TRANSPORTATION LIBRARY

Transportation
Library
TA
1
.A526

TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOLUME 95
1931

Edited by the Secretary, under the direction of the Committee on Publications.

Reprints from this publication, which is copyrighted, may be made on condition that the full title of Paper, name of Author, and page reference are given.

NEW YORK
PUBLISHED BY THE SOCIETY

1931

Entered according to Act of Congress, in the year 1931, by the American Society of Civil Engineers, in the office of the Librarian of Congress, at Washington.

Note.—This Society is not responsible for any statement made or opinion expressed in its publications.

CONTENTS

PAPERS

No. PAGE

1763 NIAGARA POWER.

By **Norman R. Gibson** 1

Discussion:

By Thomas H. Hogg 29
Frank M. Williams 34
Charles Keller 36
William Kelly 41
R. D. Johnson 43
Glen E. Edgerton 44
F. P. Williams 47
Lewis B. Stillwell 48
Norman R. Gibson 50

1764 REGULATION OF LEVELS, FLOW, AND NAVIGATION, NIAGARA RIVER: SUMMARY AND CONCLUSIONS OF VARIOUS STUDIES.

By **George B. Pillsbury** 51

Discussion:

By Robert E. Horton 63
Lynn L. Davis 75
Theron M. Ripley 79
E. W. Lane 80
F. G. Ray 81
George B. Pillsbury 83

1765 EXPERIMENTAL RESEARCH ON VIBRATION DAMPERS AND INSULATORS.

By **Stephen E. Slocum** 85

Discussion:

By J. P. Den Hartog 106
Francis G. Tatnall 108
W. M. Dawley 110
Stephen E. Slocum 125

1766 HIGH DAMS: A SYMPOSIUM 129

PAST EXPERIENCE WITH HIGH DAMS AND OUTLOOK FOR THE FUTURE.

By **A. J. Wiley** 130

CLASSIFICATION, SELECTION, AND ADAPTATION OF HIGH DAMS.

By **D. C. Henny** 139

No. PAGE

HIGH DAMS: THE VIEWPOINT OF THE GEOLOGIST.

By F. L. Ransome 149

CONSTRUCTION METHODS AND PLANT LAYOUT AT COOLIDGE DAM, IN ARIZONA.

By J. G. Tripp 159

Discussion:

By F. G. Switzer 169
Fred A. Noetzli 171
Edward Godfrey 174
B. F. Jakobsen 179
Allen Hazen 183
H. de B. Parsons 184
P. Wilhelm Werner 186
C. A. P. Turner 188
F. W. Hanna 192
H. F. Dunham 195
Lars R. Jorgensen 196
William P. Creager 197
L. F. Harza 198
A. Floris 200
Joseph Wright 205
A. J. Wiley 212
D. C. Henny 225
J. G. Tripp 230

1767 ADJUSTMENT OF TRANSIT AND STADIA TRAVERSES.

By Howard S. Rappleye 232

Discussion:

By Lowell O. Stewart 237
Paul H. Underwood 237
F. H. Peters 240
H. Parry 241
Howard S. Rappleye 243

1768 CONSTRUCTION OF THE JAMES RIVER BRIDGE PROJECT.

By R. C. Wilson and Herbert B. Pope 244

Discussion:

By Wilson T. Ballard 262
William Willoughby 266
R. C. Wilson and Herbert B. Pope 268

1769 FACTORS GOVERNING THE LOCATION OF AIRPORTS.

By Donald M. Baker 269

Discussion:

By Stephen Child 279
Morris Knowles 281
George D. Burr 283
N. H. Angell 285

No. PAGE
J. Ogden Marsh.......... 286
Archer R. Norcross.......... 287
John Nolen.......... 288
S. N. Grimm.......... 290
R. U. St. John.......... 292
John F. Partridge.......... 296
William J. Fox.......... 296
Perry A. Fellows.......... 308
W. W. Crosby.......... 308
E. K. Smith.......... 309
Clyde M. Wood.......... 311
Donald M. Baker.......... 313

1770 SPILLWAY DISCHARGE CAPACITY OF WILSON DAM.
By Louis G. Puls.......... 316

Discussion:
By Cecil E. Pearce.......... 330
J. H. Jones.......... 332
Louis G. Puls.......... 333

1771 COMPLETION OF MOFFAT TUNNEL OF COLORADO.
By Clifford Allen Betts.......... 334

Discussion:
By J. Vipond Davies.......... 372
R. H. Keays.......... 374
A. A. Eremin.......... 375
Clifford Allen Betts.......... 377

1772 RAINFALL CHARACTERISTICS AND THEIR RELATION TO SOILS AND RUN-OFF.
By C. S. Jarvis.......... 379

Discussion:
By George S. Bliss.......... 424
Brehon B. Somervell.......... 424
Jesse W. Shuman.......... 426
Clinton L. Bogert.......... 432
R. W. Powell.......... 432
Glen N. Cox.......... 439
C. R. Pettis.......... 440
George C. Danforth.......... 444
C. S. Jarvis.......... 449

1773 BALDWIN FILTRATION PLANT, CLEVELAND, OHIO.
By J. W. Ellms, G. W. Hamlin, A. G. Levy, and J. E. A. Linders.......... 459

Discussion:
By S. M. Van Loan.......... 519
Harry N. Jenks.......... 519
James H. Herron.......... 528
E. Sherman Chase.......... 529
J. W. Ellms, G. W. Hamlin, A. G. Levy, and J. E. A. Linders.... 530

No. PAGE

1774 LAMINATED ARCH DAMS WITH FORKED ABUTMENTS.

By **Fred A. Noetzli** 533

Discussion:

By LARS R. JORGENSEN 565
WILLIAM CAIN 566
B. F. JAKOBSEN 568
P. WILHELM WERNER 572
H. B. MUCKLESTON 573
FREDRIK VOGT 574
PAUL BAUMAN 583
F. W. HANNA 586
A. A. EREMIN 589
FRED A. NOETZLI 591

1775 THE SHANNON POWER DEVELOPMENT IN THE IRISH FREE STATE.

By **A. R. C. Markl** 597

Discussion:

By L. J. BEVAN 612
A. R. C. MARKL 612

1776 PLASTIC FLOW IN CONCRETE ARCHES.

By **Lorenz G. Straub** 613

Discussion:

By B. F. JAKOBSEN 679
C. A. P. TURNER 682
EDWARD GODFREY 685
J. H. GRIFFITH 688
CHARLES W. COMSTOCK 694
A. A. EREMIN 696
J. MELAN 698
FREDRIK VOGT and HERBERT J. GILKEY 699
E. PROBST 703
PAUL BAUMAN 704
G. C. STAEHLE 709
A. FLORIS 710
LORENZ G. STRAUB 711

1777 THE CHESAPEAKE AND DELAWARE CANAL.

By **Earl I. Brown** 716

Discussion:

By F. T. CHAMBERS 764

1778 EFFECT OF TURBULENCE ON THE REGISTRATION OF CURRENT METERS.

By **David L. Yarnell and Floyd A. Nagler** 766

Discussion:

By CHARLES S. BENNETT 796
CARL ROHWER 797
BREHON B. SOMERVELL 800
N. C. GROVER 806

No. PAGE

Burke L. Bigwood 811
G. H. Nettleton 812
C. Linden 814
H. R. Leach 816
Ludwig A. Ott 826
B. F. Groat 830
Forrest Nagler 834
Robert E. Horton 840
Richard Van Vliet 854
David L. Yarnell and Floyd A. Nagler 856

1779 **RELATION BETWEEN RAIL AND WATERWAY TRANSPORTATION: A SYMPOSIUM** 861

WATERWAY TRANSPORTATION FROM THE VIEWPOINT OF OPERATION.

By **T. Q. Ashburn** 862

RAILWAYS VERSUS WATERWAYS: AN ECONOMIC COMPARISON.

By **L. D. Cornish** 877

RELATION BETWEEN RAIL AND WATERWAY TRANSPORTATION.

By **E. A. Hadley** 898

Discussion:

By Lewis C. Sorrell 905
R. B. Kittredge 911
Henry A. Palmer 914
J. Parker Snow 915
Baxter L. Brown 917
Theodore Brent 919
Samuel O. Dunn 921
Henry W. Hobbs 932
C. W. Kutz 934
T. Q. Ashburn 936
L. D. Cornish 940
E. A. Hadley 946

1780 **FREQUENCY AND INTENSITY OF EXCESSIVE RAINFALLS AT BOSTON, MASSACHUSETTS.**

By **Charles W. Sherman** 951

Discussion:

By W. W. Horner 961
C. S. Jarvis 962
Milton F. Wagnitz and Lewis C. Wilcoxen 963
Charles W. Sherman 966

1781 **THE TRAINING WALL ACROSS THE LIAO BAR IN MANCHURIA.**

By **P. N. Fawcett** 969

Discussion:

By R. D. Goodrich 984
Herbert Chatley 985
H. C. Ripley 986
B. Okazaki 987
P. N. Fawcett 990

No. PAGE

1782 EVAPORATION AS A FUNCTION OF INSOLATION.
By **Burt Richardson**........ 996

Discussion:
By H. H. Kimball........ 1012
C. G. Abbot........ 1013
D. P. Krynine........ 1013
R. C. Wells........ 1016
N. W. Cummings........ 1017
Burt Richardson........ 1018

1783 HIGHWAYS AS ELEMENTS OF TRANSPORTATION.
By **Fred Lavis**........ 1020

Discussion:
By W. W. Crosby........ 1041
Walter Loring Webb........ 1042
Jay Downer........ 1044
S. Johannesson........ 1045
D. P. Krynine........ 1046
Harold M. Lewis........ 1047
Donald M. Baker........ 1053
Edwin F. Wendt........ 1054
H. W. Hudson........ 1054
Fred Lavis........ 1056

1784 SILTING AND LIFE OF SOUTHWESTERN RESERVOIRS.
By **R. G. Hemphill**........ 1060

Discussion:
By T. U. Taylor........ 1073

1785 CITY PLANNING AS RELATED TO THE SMALLER CITIES.
By **E. A. Wood**........ 1075

Discussion:
By John L. Starkie........ 1080
O. H. Koch........ 1081
Stephen Child........ 1084
E. A. Wood........ 1086

1786 FINAL REPORT OF THE SPECIAL COMMITTEE ON IMPACT IN HIGHWAY BRIDGES........ 1089

Discussion:
By Searcy B. Slack........ 1112
Llewellyn N. Edwards........ 1112
Impact in Highway Bridges Committee........ 1114

No. PAGE

1787 FINAL REPORT OF THE COMMITTEE OF THE STRUCTURAL DIVISION ON FLORIDA HURRICANE........ 1118

Discussion:

By F. O. Dufour........ 1127
T. L. Condron........ 1128
E. W. Stern........ 1128
David A. Molitor........ 1131

1788 FINAL REPORT OF COMMITTEE OF POWER DIVISION ON ICE AS AFFECTING POWER PLANTS........ 1134

Discussion:

By C. M. Daily........ 1151

1789 SECOND PROGRESS REPORT OF THE SPECIAL COMMITTEE ON STEEL COLUMN RESEARCH........ 1152

Discussion:

By Henry S. Prichard........ 1255
E. H. Salmon........ 1258
J. H. Griffith........ 1267

1790 FINAL REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION ON SEWAGE WORKS OPERATION AND CONTROL........ 1273

1791 THE ENGINEER'S GROWING CIVIC RESPONSIBILITIES: ADDRESS AT THE ANNUAL CONVENTION, TACOMA, WASHINGTON, JULY 8, 1931.

By Francis Lee Stuart........ 1294

1792 REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION ON "A NATIONAL RECLAMATION POLICY"........ 1303

A NATIONAL RECLAMATION POLICY: EXPLANATORY STATEMENT REGARDING REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION.

By J. B. Lippincott........ 1307

A NATIONAL RECLAMATION POLICY: ECONOMIC ASPECTS OF FEDERAL RECLAMATION.

By Elwood Mead........ 1321

Discussion:

By George W. Fuller........ 1328
W. F. Russell........ 1329
Nils A. Olsen........ 1331
L. C. Gray........ 1331
Frank Adams........ 1331
M. J. Dowd........ 1335
C. E. Grunsky........ 1338
S. T. Harding........ 1341
H. H. Dare........ 1343
Charles R. Hedke........ 1346
Charles Kirby Fox........ 1357
Rhodes E. Rule........ 1360

PAGE
George M. Bacon.... 1362
G. E. P. Smith.... 1364
R. K. Tiffany.... 1371
David Weeks.... 1373
John A. Widtsoe.... 1381
F. H. Newell.... 1383
A. D. Lewis.... 1388
Joseph Jacobs.... 1392
F. N. Cronholm.... 1395
R. I. Meeker.... 1401
E. B. Debler.... 1403
John E. Field.... 1408
National Reclamation Policy Committee.... 1416

MEMOIRS OF DECEASED MEMBERS

Clemens Herschel, Past-President and Hon. M. Am. Soc. C. E.... 1419
George Smedley Webster, Past-President, Am. Soc. C. E.... 1423
George Davidson, Hon. M. Am. Soc. C. E.... 1428
Grenville Mellen Dodge, Hon. M. Am. Soc. C. E.... 1432
Kenneth Allen, M. Am. Soc. C. E.... 1437
William Franklin Allison, M. Am. Soc. C. E.... 1440
Justin Kenderdine Anderson, M. Am. Soc. C. E.... 1442
Henry Clarke Ash, M. Am. Soc. C. E.... 1443
Frank Milligan Ashmead, M. Am. Soc. C. E.... 1445
Frank Foy Axtell, M. Am. Soc. C. E.... 1446
Mortimer Grant Barnes, M. Am. Soc. C. E.... 1449
Axel Samuel Frederick Berquist, M. Am. Soc. C. E.... 1451
Alexander Leslie Black, M. Am. Soc. C. E.... 1452
John Carlisle Bland, M. Am. Soc. C. E.... 1453
George Bowers, M. Am. Soc. C. E.... 1456
Stoerk Johan Bratager, M. Am. Soc. C. E.... 1457
Peter Franklin Brendlinger, M. Am. Soc. C. E.... 1459
Robert Brown, M. Am. Soc. C. E.... 1462
Henry Amsden Burr, M. Am. Soc. C. E.... 1463
Maxwell Cunningham Byers, M. Am. Soc. C. E.... 1465
William Cain, M. Am. Soc. C. E.... 1467
George Linden Campen, M. Am. Soc. C. E.... 1473
William Waller Carson, M. Am. Soc. C. E.... 1475
Maurice Wurtz Cooley, M. Am. Soc. C. E.... 1477
Edwin Duryea, Jr., M. Am. Soc. C. E.... 1478
Harrison Robert Fehr, Sr., M. Am. Soc. C. E.... 1480
Samuel Morse Felton, M. Am. Soc. C. E.... 1481
Roger Morse Freeman, M. Am. Soc. C. E.... 1485
Horace Delphos Gates, M. Am. Soc. C. E.... 1488
Thomas Golding Gerdine, M. Am. Soc. C. E.... 1490
Thomas Nixon Gilmore, M. Am. Soc. C. E.... 1492
Benjamin Mortimer Hall, M. Am. Soc. C. E.... 1493
Alfred Ernest Harvey, M. Am. Soc. C. E.... 1496
Harry Hawgood, M. Am. Soc. C. E.... 1497
Allen Hazen, M. Am. Soc. C. E.... 1500
Albert Banks Hill, M. Am. Soc. C. E.... 1503
John Wilmuth Hill, M. Am. Soc. C. E.... 1506
Albert Harrison Hogeland, M. Am. Soc. C. E.... 1509
Henry Holgate, M. Am. Soc. C. E.... 1511
Richard Leveridge Hoxie, M. Am. Soc. C. E.... 1513
William Hausmer Hoyt, M. Am. Soc. C. E.... 1515

PAGE
Marion Columbus Huckaby, M. Am. Soc. C. E. 1517
Hector James Hughes, M. Am. Soc. C. E. 1518
John Edward Jennings, M. Am. Soc. C. E. 1520
Peter Junkersfeld, M. Am. Soc. C. E. 1522
Thomas Pearson Kinsley, M. Am. Soc. C. E. 1524
George Charles Kreutzer, M. Am. Soc. C. E. 1526
King Yang Kwong, M. Am. Soc. C. E. 1528
Eugene McLean Long, M. Am. Soc. C. E. 1531
Henry Howell Lotter, M. Am. Soc. C. E. 1533
Jacob Lott Ludlow, M. Am. Soc. C. E. 1536
John Lundie, M. Am. Soc. C. E. 1538
Henry Martyn MacKay, M. Am. Soc. C. E. 1541
William Walter Maclay, M. Am. Soc. C. E. 1543
George Arnold McCarthy, M. Am. Soc. C. E. 1545
William Edward McClintock, M. Am. Soc. C. E. 1546
Laurence Francis McCoy, M. Am. Soc. C. E. 1549
John Alexander McDonald, M. Am. Soc. C. E. 1550
Anson Burlingame McGrew, M. Am. Soc. C. E. 1552
Clark Rogers Mandigo, M. Am. Soc. C. E. 1553
Marsden Manson, M. Am. Soc. C. E. 1554
Charles Cyril Martin, M. Am. Soc. C. E. 1556
Sheldon Elton Minor, M. Am. Soc. C. E. 1558
Henry Mitchell, M. Am. Soc. C. E. 1560
John Mitchell Moncrieff, M. Am. Soc. C. E. 1561
Guy Moulton, M. Am. Soc. C. E. 1563
Andreas Wendelbo Münster, M. Am. Soc. C. E. 1565
Ralph Waldo Nickerson, M. Am. Soc. C. E. 1566
George H. Norman, M. Am. Soc. C. E. 1568
George Harvey Norton, M. Am. Soc. C. E. 1569
Charles Wellman Parks, M. Am. Soc. C. E. 1571
James Suydam Polhemus, M. Am. Soc. C. E. 1575
James Bond Pope, M. Am. Soc. C. E. 1576
George Frederick Porter, M. Am. Soc. C. E. 1577
William Abbott Pratt, M. Am. Soc. C. E. 1579
Marshall Rogers Pugh, M. Am. Soc. C. E. 1581
John William Raitt, M. Am. Soc. C. E. 1584
William Boardman Reed, M. Am. Soc. C. E. 1585
Claude Irvin Rhodes, M. Am. Soc. C. E. 1586
Franklin Riffle, M. Am. Soc. C. E. 1587
Edward Austin Rix, M. Am. Soc. C. E. 1590
Thomas Rodd, M. Am. Soc. C. E. 1592
Hans Henrik Rode, M. Am. Soc. C. E. 1594
John Henry Rostock, M. Am. Soc. C. E. 1595
Harry Harwood Rousseau, M. Am. Soc. C. E. 1597
Richard Lord Russell, M. Am. Soc. C. E. 1601
George Frederick Samuel, M. Am. Soc. C. E. 1603
Ivan Oscar Shaffer, M. Am. Soc. C. E. 1604
Stanley Rush Sharts, M. Am. Soc. C. E. 1605
Bronson Hasbrouck Smith, M. Am. Soc. C. E. 1606
Linton Waddell Stubbs, M. Am. Soc. C. E. 1608
Harry Taylor, M. Am. Soc. C. E. 1609
Louis Lincoln Tribus, M. Am. Soc. C. E. 1612
Edmund Kimball Turner, M. Am. Soc. C. E. 1614
Charles Augustus Van Keuren, M. Am. Soc. C. E. 1615
Robert Lawrence Van Sant, M. Am. Soc. C. E. 1616
Harry Lee Van Zile, M. Am. Soc. C. E. 1617
Edward Franklin Vincent, M. Am. Soc. C. E. 1619
Albert Lowry Webster, M. Am. Soc. C. E. 1621
James Edward Whitfield, M. Am. Soc. C. E. 1622
Frank Clinton Wight, M. Am. Soc. C. E. 1623
Frank Martin Williams, M. Am. Soc. C. E. 1626

PAGE
William Fish Williams, M. Am. Soc. C. E. 1628
Carlton Carpenter Witt, M. Am. Soc. C. E. 1630
Morgan Edward Yeatman, M. Am. Soc. C. E. 1632
Samuel McCain Young, M. Am. Soc. C. E. 1633
Raymond Arden Edwards, Assoc. M. Am. Soc. C. E. 1634
William James Fulton, Assoc. M. Am. Soc. C. E. 1636
Arthur Bates Goodwin, Assoc. M. Am. Soc. C. E. 1637
Charles Ellett Hart, Assoc. M. Am. Soc. C. E. 1639
Robert Grier Hemphill, Assoc. M. Am. Soc. C. E. 1639
Edward Baxter Hill, Assoc. M. Am. Soc. C. E. 1641
Edwin Allston Hill, Assoc. M. Am. Soc. C. E. 1642
Cornelius Joseph O'Connor, Assoc. M. Am. Soc. C. E. 1644
Waldo Justin Bickle Porter, Assoc. M. Am. Soc. C. E. 1645
Joseph Warren Rogers, Assoc. M. Am. Soc. C. E. 1646
Nicholas Alexander Scripko, Assoc. M. Am. Soc. C. E. 1649
William Laurie Seaman, Assoc. M. Am. Soc. C. E. 1650
Erastus Roland Simpson, Assoc. M. Am. Soc. C. E. 1652
Harold David Stoll, Assoc. M. Am. Soc. C. E. 1653
Franklin Stevens Storey, Assoc. M. Am. Soc. C. E. 1655
John Gerard Theban, Assoc. M. Am. Soc. C. E. 1656
Walter Checkley Tiffany, Assoc. M. Am. Soc. C. E. 1658
William Franklin Whitaker, Assoc. M. Am. Soc. C. E. 1660
Robert Elgene Yolton, Assoc. M. Am. Soc. C. E. 1661
James Russell Vaughan, Jun. Am. Soc. C. E. 1662
Stedman Bent, Affiliate, Am. Soc. C. E. 1663
Clarence Marvin Foster, Affiliate, Am. Soc. C. E. 1664

ERRATUM

Transactions Vol. 92

Page 1516, Equation (105), for "$- X r_n \left(\frac{1 - \sin \phi_1}{\phi_1}\right)$", read, "$- X r_n \left(1 - \frac{\sin \phi_1}{\phi_1}\right)$".

PAPERS AND REPORTS OF SPECIAL COMMITTEES PUBLISHED IN PROCEEDINGS, AM. SOC. C. E., OCTOBER, 1929, TO MAY, 1931, INCLUSIVE, NOT PUBLISHED IN TRANSACTIONS VOL. 95.

Number of *Proceedings.*
REPORT OF THE COMMITTEE ON ESSENTIAL FACTS CONCERNING THE FAILURE OF THE ST. FRANCIS DAM .... October, 1929*
THE BEHAVIOR OF A REINFORCED CONCRETE ARCH DURING CONSTRUCTION.
By Searcy B. Slack .... November, 1929*
GENERAL SPECIFICATIONS FOR STEEL RAILWAY BRIDGES: PREPARED BY CONFERENCE COMMITTEES FROM THE AMERICAN SOCIETY OF CIVIL ENGINEERS AND THE AMERICAN RAILWAY ENGINEERING ASSOCIATION .... December, 1929*

---

* Published in *Proceedings* only.

Number of *Proceedings.*

SOME ASPECTS OF WATER CONSERVATION.
By R. A. Sutherland........September, 1930

CONSTRUCTION OF LA OLA PIPE LINE IN CHILE.
By W. B. Saunders........September, 1930

TESTS OF BROAD-CRESTED WEIRS.
By James G. Woodburn........September, 1930

STRESSES IN GRAVITY DAMS BY PRINCIPLE OF LEAST WORK.
By B. F. Jakobsen........September, 1930

THEORY OF SIMILARITY AND MODELS.
By Benjamin F. Groat........October, 1930

FORMULAS FOR RAINFALL INTENSITIES OF LONG DURATION.
By Merrill M. Bernard........October, 1930

PROGRESS REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION ON FILTERING MATERIALS FOR WATER AND SEWAGE WORKS........October, 1930*

THE BUTTRESSED DAM OF UNIFORM STRENGTH.
By Herman Schorer........November, 1930

ECONOMICAL DESIGN OF HYDRAULIC PIPE-LINE DREDGE.
By John Francis Cushing........November, 1930

HIGHWAY LOCATION: GENERAL CONSIDERATIONS.
By Gibb Gilchrist........November, 1930

HIGHWAY LOCATION: PRACTICAL CONSIDERATIONS.
By A. R. Losh........November, 1930

RELOCATING LAND SURVEYS IN TEXAS.
By B. F. Williams........November, 1930

FLOW OF WATER IN TIDAL CANALS.
By Earl I. Brown........December, 1930

THE DON MARTIN PROJECT.
By Andrew Weiss........December, 1930

DESIGN OF A REINFORCED CONCRETE SKEW ARCH.
By Bernard L. Weiner........January, 1931

A DISCHARGE DIAGRAM FOR UNIFORM FLOW IN OPEN CHANNELS.
By Murray Blanchard........January, 1931

LEGAL PROBLEMS INVOLVED IN ESTABLISHING SET-BACK LINES.
By Clifton Williams........January, 1931

THE EIGHT-MILE CASCADE TUNNEL, GREAT NORTHERN RAILWAY: A SYMPOSIUM........February, 1931

MEASURING IRRIGATION DELIVERIES IN THE PANJAB.
By E. S. Lindley........February, 1931

SURVEYS FOR SWIFT RIVER RESERVOIR OF THE BOSTON METROPOLITAN WATER SUPPLY.
By N. LeRoy Hammond........February, 1931

---

* Published in *Proceedings* only.

Number of *Proceedings*.

FLOW OF GROUND-WATER AS APPLIED TO DRAINAGE WELLS.
By M. R. Lewis.. March, 1931

GRAVITY DAMS ARCHED DOWN STREAM.
By B. J. Lambert.. March, 1931

AN ANALYSIS OF MULTIPLE-SKEW ARCHES ON ELASTIC PIERS.
By J. Charles Rathbun.................................. April, 1931

RUN-OFF—RATIONAL RUN-OFF FORMULAS.
By R. L. Gregory and C. E. Arnold........................ April, 1931

MANUFACTURING CONCRETE OF UNIFORM QUALITY.
By William M. Hall...................................... May, 1931

SPECIFICATIONS FOR BOUNDARY SURVEYS: PREPARED BY THE COMMITTEE OF THE SURVEYING AND MAPPING DIVISION ON BOUNDARY SURVEYS.................................... May, 1931*

---

* Published in *Proceedings* only.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 1763

NIAGARA POWER[1]

BY NORMAN R. GIBSON,[2] M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. THOMAS H. HOGG, FRANK M. WILLIAMS, CHARLES KELLER, WILLIAM KELLY, R. D. JOHNSON, GLEN E. EDGERTON, F. P. WILLIAMS, LEWIS B. STILLWELL, AND NORMAN R. GIBSON

SYNOPSIS

In preparing this paper it has seemed best to present, first, in barest outline, a general picture of the engineering phases of the power plants at Niagara Falls, illustrating their principal characteristics and tracing their development as the art has progressed; next, to discuss some of the newer problems connected with power in the Niagara River; and, finally, to direct attention to the significance of Niagara power in the industrial and social development of two extensive and important territories in two countries. In order to avoid needless repetition, detail descriptions of the plants are not given, but instead a classified bibliography is appended for such further information as may be desired.

EARLY POWER PLANTS

The small power plants built early in the Nineteenth Century have long since disappeared, leaving but little evidence of their one-time existence. Power development at Niagara Falls emerged from the dim past about 1853, when Messrs. Caleb S. Woodhull, Walter Bryant, and their associates commenced the construction of the hydraulic canal, about one mile long and extending from Port Day on the American side of the river just above the Falls to a point on the Gorge about a mile below the Falls. This canal, when enlarged to its present dimensions, was destined to become a part of one of the most important plants existing to-day. It was first put into operation

---

[1] Presented at the meeting of the Power Division, Buffalo, N. Y., July 19, 1928.

[2] Vice-Pres. and Chf. Engr., Buffalo, Niagara & Eastern Power Corporation, Buffalo, N. Y.

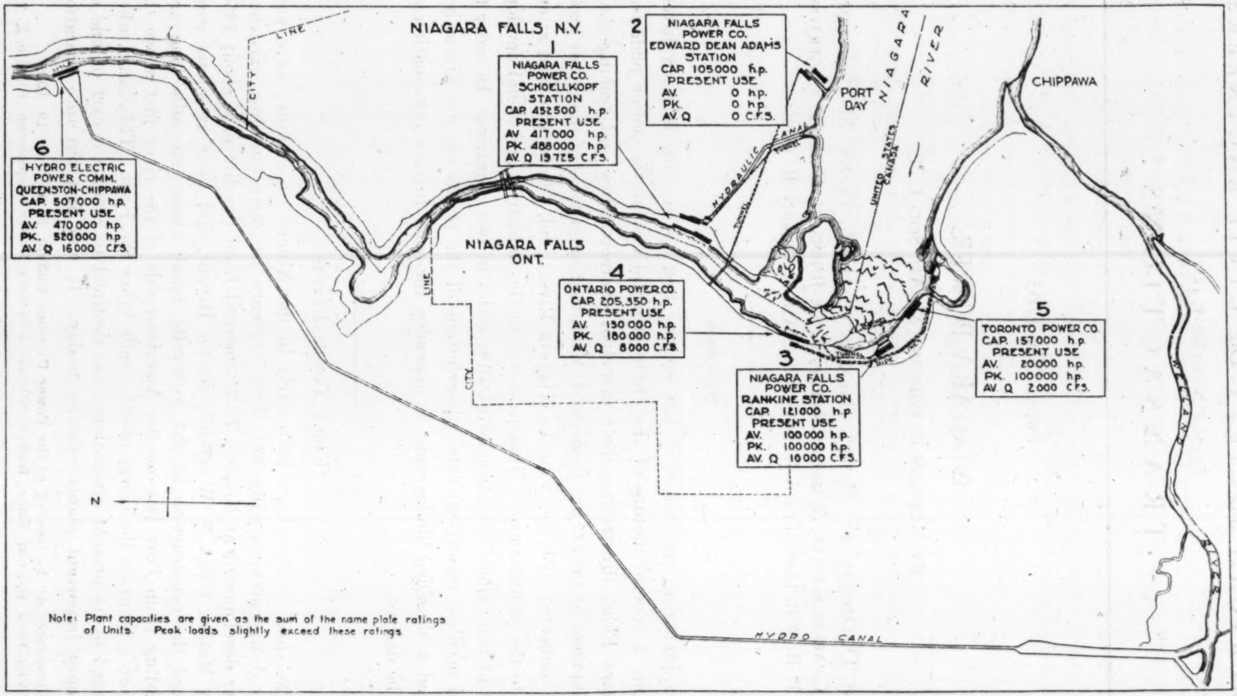
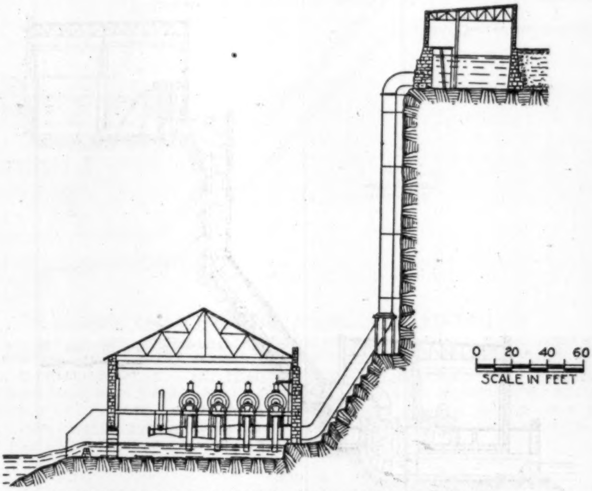

NIAGARA FALLS N.Y.
1
NIAGARA FALLS POWER CO. SCHOELLKOPF STATION
CAP. 452500 h.p.
PRESENT USE
AV 417000 h.p.
PK. 488000 h.p.
AV. Q 19725 C.F.S.
2
NIAGARA FALLS POWER CO. EDWARD DEAN ADAMS STATION
CAP. 105000 h.p.
PRESENT USE
AV. 0 h.p.
PK. 0 h.p.
AV. Q 0 C.F.S.
3
NIAGARA FALLS POWER CO. RANKINE STATION
CAP. 121000 h.p.
PRESENT USE
AV. 100000 h.p.
PK. 100000 h.p.
AV. Q 10000 C.F.S.
4
ONTARIO POWER CO.
CAP. 205,350 h.p.
PRESENT USE
AV. 130000 h.p.
PK. 180000 h.p.
AV. Q 8000 C.F.S.
5
TORONTO POWER CO.
CAP. 157000 h.p.
PRESENT USE
AV. 20000 h.p.
PK. 100000 h.p.
AV. Q 2000 C.F.S.
6
HYDRO ELECTRIC POWER COMM. QUEENSTON-CHIPPAWA
CAP. 507000 h.p.
PRESENT USE
AV. 470000 hp
PK. 520000 hp
AV. Q 16000 C.F.S.
NIAGARA FALLS ONT.
PORT DAY
NIAGARA RIVER
UNITED STATES
CANADA
CHIPPAWA
WELLAND RIVER
HYDRO CANAL
HYDRAULIC CANAL
TUNNEL
PIPE LINES
CITY LINE
N
Note: Plant capacities are given as the sum of the name plate ratings of Units. Peak loads slightly exceed these ratings.

FIG. 1.—MAP SHOWING LOCATION OF EXISTING NIAGARA POWER PLANTS.

about 1875, when it had a cross-section varying from 20 to 36 ft. in width and about 8 to 10 ft. in depth. Since then it has been used continuously and enlarged from time to time until it has now reached a size of 100 ft. wide by about 20 ft. deep.

Present Power Plants

To-day (1928), there are six major hydro-electric generating stations at Niagara Falls, having a total installed capacity of 1 547 850 h.p. As shown in Fig. 1, four of these stations, aggregating an installed capacity of 990 350 h.p., are on the Canadian side of the river, and two, with an installation of 557 500 h.p., are on the American side. The ownership and control of all these plants are vested in two organizations, the two plants on the American side and one of those in Canada being owned or controlled by The Niagara Falls Power Company, a New York corporation, and the remaining three plants on the Canadian side being under the Hydro-Electric Power Commission of Ontario.



THE NIAGARA FALLS POWER CO. SCHOELLKOPF STATION 2

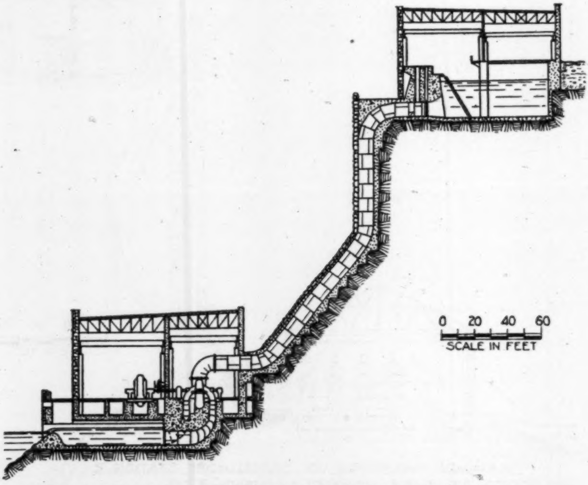
CONSTRUCTED: 1895 TO 1904 ABANDONED: 1923 DEMOLISHED: 1925.
INSTALLATION: (1921) 4-UNITS 2300 H.P., 4-2900 H.P., 1-2800 H.P., 250 R.P.M. TOTAL 9 UNITS, 23600 H.P.
TURBINES: HORIZONTAL SHAFT, DOUBLE RUNNER, GEYLIN, JONVAL AND FRANCIS TYPES.
GENERATORS: EARLIER INSTALLATIONS INCLUDED A.C. UNITS AT 11000 VOLT, 3 PHASE, 30 CYCLES AND 2 200 VOLT SINGLE PHASE 125 CYCLES. (1921) 4-750 KW. 300 VOLT D.C., 2-875 KW. 180 VOLT D.C., 8-1000 KW. 300 VOLT D.C. 4-750 KW. 350 VOLT D.C. TWO GENERATORS DIRECTLY COUPLED TO TURBINE SHAFT. NET HEAD: ABOUT 210 FT. UNIT EFFICIENCY: 80%.
PENSTOCKS: PLATE STEEL, 1-8'-0" DIAM. 240 FT. LONG, 2-11'-0" DIAM. 270 FT. LONG. SLIDING STEEL HEAD GATES. WATER SUPPLY: CANAL 100 FT. WIDE BY 20 FT. DEEP, AND 4 500 FT. LONG FROM INTAKE ABOUT ONE MILE ABOVE FALLS. SUPPLYING ALSO STATIONS 3A AND 3B.

Fig. 2

Schoellkopf Station.—The hydraulic canal, Fig. 1, was purchased in 1877 by Jacob F. Schoellkopf and others who, a year later, organized The Niagara Falls Hydraulic Power and Manufacturing Company, using the canal to supply water to wheels in mills located at the top of the high bank along the Gorge. The water-wheels were placed in the bottom of shafts or pits sunk beneath the mills at various depths, and utilized from 25 to 86 ft. of the total 215 ft. available head.

Following early experimental stations from 1880 onward, The Niagara Falls Hydraulic Power and Manufacturing Company constructed, in 1881, its first commercial hydro-electric generating station, which was known as Station No. 1. It had a capacity of 2 000 h.p. generated by direct-current dynamos operated by rope drives from water-wheels. This station was abandoned in 1904.

Station No. 2 of this Company (Fig. 2) was begun in 1895, completed about 1904, abandoned in 1921, and demolished in 1925. Originally, it had a total of fifteen hydro-electric units with an aggregate capacity of 33 900 h.p. These units were supplied with water from the hydraulic canal by three steel-plate penstocks, each of which fed from four to six units. In 1914, six units and one penstock were abandoned, leaving nine units having an aggregate capacity of 23 600 h.p.



THE NIAGARA FALLS POWER CO. SCHOELLKOPF STATION 3A

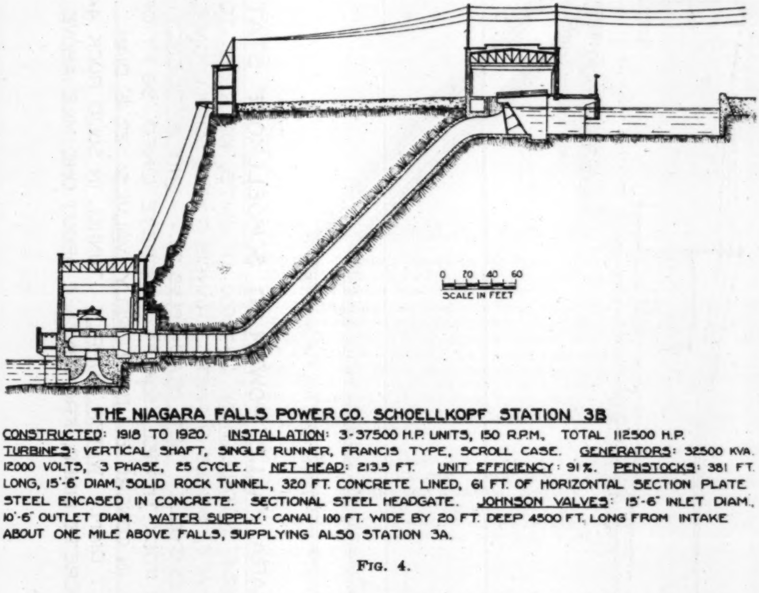
CONSTRUCTED: 1905 TO 1914. INSTALLATION: 13-10000 H.P. UNITS, 300 R.P.M., TOTAL 130000 H.P.
TURBINES: HORIZONTAL SHAFT, DOUBLE RUNNER, FRANCIS TYPE, SCROLL CASE.
GENERATORS: 8000 KVA., 12000 VOLTS, 3 PHASE, 25 CYCLES. NET HEAD: 210 FT. UNIT EFFICIENCY: 86%.
PENSTOCKS: PLATE STEEL, 260 FT. LONG, 9'-0" DIAM., GATE VALVES. WATER SUPPLY: CANAL 100 FT. WIDE BY 20 FT. DEEP, 4500 FT. LONG FROM INTAKE ABOUT ONE MILE ABOVE FALLS, SUPPLYING ALSO STATION 3B.

FIG. 3.

About 1903, The Niagara Falls Hydraulic Power and Manufacturing Company began the construction of Section A of its Station No. 3 (Fig. 3), to contain thirteen 10 000-h.p. units. This, together with the addition of Section B (Fig. 4), having three 37 500-h.p. units constructed during the years 1918 to 1920, and Section C (Fig. 5), with three 70 000-h.p. units constructed in the period, 1922 to 1924, constitutes the largest and most important plant on the American side to-day. Its aggregate installed capacity is 452 500 h.p.

The hydraulic canal which, with frequent enlargements, had been able to supply water to all the plants of this Company, reached the limit of its capacity after the addition of Station No. 3-B. A tunnel, 32 ft. in diameter, from Port Day to the Canal Basin, was constructed, therefore, as a part of the Station No. 3-C addition. In 1927, Station No. 3 was named Schoellkopf Station in honor of Jacob F. Schoellkopf and his family of which three generations have devoted themselves to the task of developing Niagara power.

In 1910, The Niagara Falls Hydraulic Power and Manufacturing Company became the Hydraulic Power Company of Niagara Falls, and, in 1918, the latter was consolidated with The Niagara Falls Power Company.

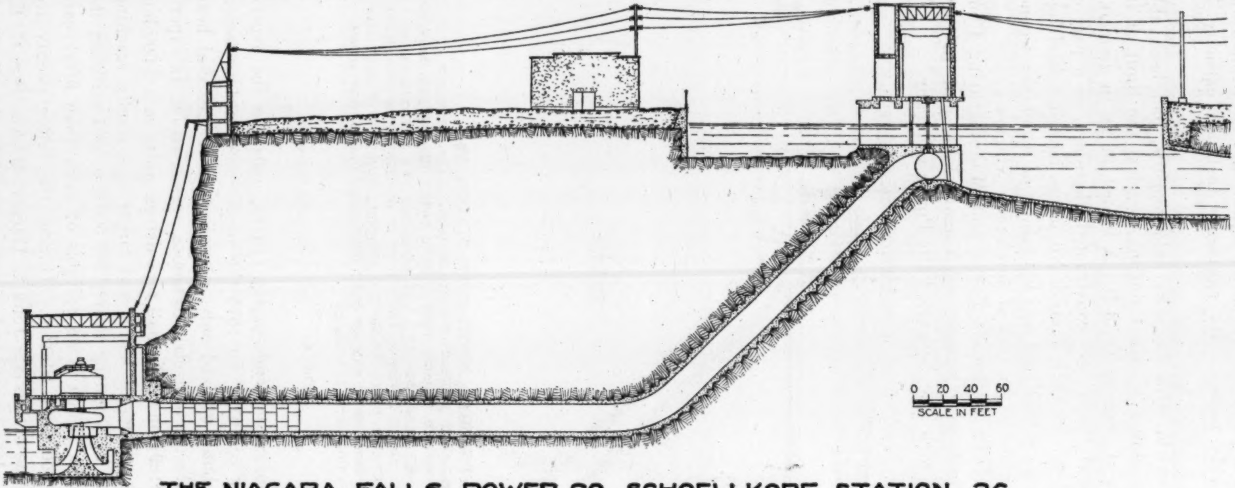


THE NIAGARA FALLS POWER CO. SCHOELLKOPF STATION 3B

CONSTRUCTED: 1918 TO 1920. INSTALLATION: 3-37500 H.P. UNITS, 150 R.P.M., TOTAL 112500 H.P. TURBINES: VERTICAL SHAFT, SINGLE RUNNER, FRANCIS TYPE, SCROLL CASE. GENERATORS: 32500 KVA. 12000 VOLTS, 3 PHASE, 25 CYCLE. NET HEAD: 213.5 FT. UNIT EFFICIENCY: 91%. PENSTOCKS: 381 FT. LONG, 15'-6" DIAM., SOLID ROCK TUNNEL, 320 FT. CONCRETE LINED, 61 FT. OF HORIZONTAL SECTION PLATE STEEL ENCASED IN CONCRETE. SECTIONAL STEEL HEADGATE. JOHNSON VALVES: 15'-6" INLET DIAM., 10'-6" OUTLET DIAM. WATER SUPPLY: CANAL 100 FT. WIDE BY 20 FT. DEEP 4500 FT. LONG FROM INTAKE ABOUT ONE MILE ABOVE FALLS, SUPPLYING ALSO STATION 3A.

Fig. 4.

The Schoellkopf Station, as it stands to-day (1928), embodies the experience and progress of half a century. In 1875 the small canal feeding waterwheels in mills along the bank utilized only a small part of the total head available. After 1880 the then new electric dynamos were tried out in experimental stations, and from 1891 onward small rope-driven units in a 2 000-h.p. station were followed by machines of larger and larger size, finally reaching 70 000 h.p. each in a station aggregating nearly 500 000 h.p. and utilizing the full 215 ft. of head available. Such is an epitome of more than fifty years of development in the Schoellkopf Station. During the latter twenty-five years of that period, the name of the late John L. Harper, M. Am. Soc. C. E., stands out as responsible in large measure for such successful achievement.

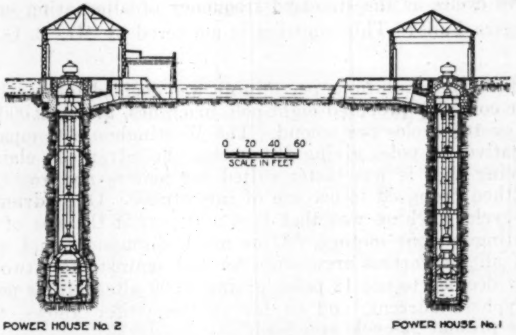


THE NIAGARA FALLS POWER CO. SCHOELLKOPF STATION 3C

CONSTRUCTED: 1921 TO 1924. INSTALLATION: 3 - 70000 H.P. UNITS, 107 R.P.M. TOTAL 210000 H.P. TURBINES: VERTICAL SHAFT, SINGLE RUNNER, FRANCIS TYPE, SCROLL CASE. GENERATORS: 65000 KVA. 12000 VOLTS, 3 PHASE, 25 CYCLE. NET HEAD: 213.5 FT. UNIT EFFICIENCY: 92 %. PENSTOCKS: 600 FT. LONG, 21'-0" DIAM., SOLID ROCK TUNNEL, 504 FT. CONCRETE LINED, 96 FT. OF HORIZONTAL SECTION PLATE STEEL ENCASED IN CONCRETE. BUTTERFLY VALVES: 23'-6" DIAM. JOHNSON VALVES: 21'-0" INLET DIAM., 14'-0" OUTLET DIAM. WATER SUPPLY: TUNNEL IN SOLID ROCK 4400 FT. LONG, 32'-0" DIAM. HORSESHOE SHAPE, CONCRETE LINED, FROM INTAKE ABOUT ONE MILE ABOVE FALLS. TUNNEL AND CANAL INTERCONNECTED.

FIG. 5.

Edward Dean Adams Station.—The Niagara Falls Power Company was created in 1886, under the name of The Niagara River Hydraulic Tunnel Power and Sewer Company, the name being changed in 1889. At that time it was debated whether the Company should develop hydraulic power to be delivered to mills located along its waterways, or pneumatic power for transmission to industrial districts and even to Buffalo, N. Y. This question was settled by the rapid development of electrical power which definitely displaced both the other proposals.



THE NIAGARA FALLS POWER CO. EDWARD DEAN ADAMS STATION.

POWER HOUSE No. 1

CONSTRUCTED: 1890 TO 1900. TURBINES REMODELED 1910 TO 1912. INSTALLATION: 10-5000 H.P. UNITS, 250 R.P.M., TOTAL 50000 H.P. TURBINES: VERTICAL SHAFT, SINGLE RUNNER, FRANCIS TYPE. CYLINDRICAL GATES. GENERATORS: 5-4400 KVA. 12000 VOLT 3 PHASE, 25 CYCLES, 5-4400 KVA. 2200 VOLT 2 PHASE 25 CYCLES NET HEAD: 135 FT. UNIT EFFICIENCY: 78 % SHAFT: TUBULAR 3'-2" DIAM. PENSTOCKS: PLATE STEEL 7'-6" DIAM., 144 FT. LONG, SLIDING STEEL HEAD GATES. WATER SUPPLY: INTAKE CANAL ABOUT ONE AND A QUARTER MILES ABOVE FALLS, WIDTH 119 FT. TO 194 FT. AVERAGE DEPTH 12 FT., 1200 FT. LONG, SUPPLIES POWER HOUSE No 2 ALSO. DISCHARGE TUNNEL: HORSESHOE SHAPE, BRICK LINED, 21 FT. HIGH BY 18'-10" WIDE, 7481 FT. LONG. SAME TUNNEL DISCHARGES WATER FROM POWER HOUSE No. 2.

POWER HOUSE No. 2

CONSTRUCTED 1900 TO 1904 INSTALLATION: 11-5000 H.P. UNITS, 250 R.P.M. TOTAL 55000 H.P. TURBINES: VERTICAL SHAFT, SINGLE RUNNER, FRANCIS TYPE, CYLINDRICAL GATES. GENERATORS: 11-4400 KVA. 2200 VOLT 2 PHASE 25 CYCLES NET HEAD: 135 FT. UNIT EFFICIENCY: 73 % SHAFT: TUBULAR 3'-2" DIAM. PENSTOCKS: PLATE STEEL 7'-6" DIAM., 148 FT. LONG, SLIDING STEEL HEAD GATES. WATER SUPPLY AND DISCHARGE TUNNEL SAME AS POWER HOUSE No. 1

Fig. 6.

After much consideration a new type of plant was designed for the Edward Dean Adams Station. In this the water-wheels were placed in the bottom of a wheel-pit, about 24 ft. wide and 130 ft. deep, excavated in the solid rock. The discharge from the wheels was carried to the river through a flow-line tunnel about 7 000 ft. long, which discharged into the Gorge about ½ mile below the Falls. Long vertical shafts connected the water-wheels to the generators which were placed in a power house above ground.

By 1895, alternating-current electrical power was being generated in Station No. 1 and delivered in Niagara Falls. Power was sent to Tonawanda,

N. Y., and Buffalo in 1896 by means of 11 000-volt transmission. By 1900, Station No. 1 had been completed with ten units of 5 000 h.p. each, and, by 1904, Station No. 2 had been completed with a further installation of eleven units of 5 000 h.p. each. Fig. 6 shows a typical cross-section of both stations. On the completion of Section C of the Schoellkopf Station in 1924, the twenty-one units aggregating 105 000 h.p. in the Adams Station were placed in reserve.

One of the questions most frequently asked is, "What led to the adoption of twenty-five cycles as the standard frequency of alternating current generated at Niagara Falls?" This question is answered by Mr. B. G. Lamme,[3] as follows:

> "A speed of 250 revolutions per minute was decided upon. The engineers of the power company proposed eight-pole machines, giving 2 000 alternations per minute or 16⅔ cycles per second. The Westinghouse Company proposed, as an alternative, 16 poles giving 33⅓ cycles, the advantage claimed for this frequency being that it was better suited for motors and rotary converters, which were then promised to become of importance. One advantage claimed for the 16⅔-cycle machine was that it would permit the use of commutator type alternating-current motors. After much discussion, and weighing and balancing of all the various arguments for and against these two frequencies, it was finally decided to use 12 poles, giving 3 000 alternations per minute, or 25-cycle polyphase current, and as far as the writer knows, this was the origin of the present 25-cycle standard."

This station is named in honor of Edward Dean Adams, F. Am. Soc. C. E., who took a foremost part in the promotion of the undertaking and has been actively associated with The Niagara Falls Power Company since its inception.

William B. Rankine Station.—About 1900, when the Edward Dean Adams Station No. 1 was nearing completion, the management of The Niagara Falls Power Company turned its attention toward the development of power on the Canadian side of the river, for which it had obtained rights from the Government of the Province of Ontario in 1892. By this time sufficient experience had been gained to warrant the construction of hydro-electric generating units of 10 000 h.p. capacity. The wheel-pit and tail-race tunnel type of plant was again adopted in the design of this station (Fig. 7). By 1905, power was being delivered from this Canadian plant and, by 1917, ten units had been installed having an aggregate capacity of 109 000 h.p. In 1924, Unit No. 11, of 12 000 h.p. capacity, was added, thus bringing the total installed capacity of the station up to 121 000 h.p.

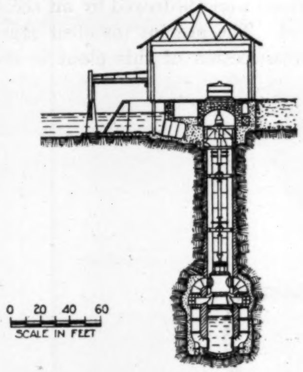
This station is named in honor of William B. Rankine, who perhaps more than any other person bore the burden of the pioneer work in The Niagara Falls Power Company.

International Railway Company.—A small installation on the Canadian side of the river, a short distance above the Horseshoe Falls, was begun in 1892 and completed with a total installation of 3 600 h.p. in 1905. This plant is owned by the International Railway Company and is used to generate direct current for the operation of the railway.

---

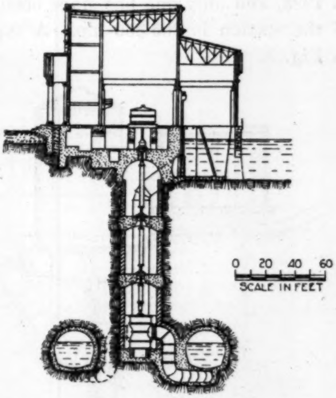
[3] "Electrical Engineering Papers," p. 602.

Toronto Power Company.—Power projects at Niagara Falls began to multiply early in the Twentieth Century and two more plants were constructed on the Canadian side of the river. One of these was developed by the Electrical Development Company, now commonly referred to as the Toronto Power Company, a confusion which originated by substituting the name of the transmission company for that of the developing company. This Company followed somewhat the same lines of development as were used for the Adams and Rankine Stations, but brought the discharge end of its tail-race tunnel through the cliff directly behind the sheet of water falling over the Horseshoe Falls.



THE NIAGARA FALLS POWER COMPANY
WILLIAM BIRCH RANKINE STATION
CONSTRUCTED: 1901 TO 1924. INSTALLATION: 5 UNITS 10250 H.P., 2-12750 H.P., 3-10750 H.P., 1-12000 H.P., 250 R.P.M., TOTAL 11 UNITS 121000 H.P. TURBINES: VERTICAL SHAFT, FRANCIS TYPE, 7 DOUBLE RUNNER AND 4 SINGLE RUNNER, CYLINDRICAL GATES IN ALL BUT 12000 H.P UNIT WHICH HAS WICKET GATES. GENERATORS: 5-8800 KVA., 5-10400 KVA., 1-11440 KVA., 12000 VOLT, 3 PHASE, 25 CYCLES. NET HEAD: 135 FT. UNIT EFFICIENCY: 70%. SHAFT: TUBULAR 3'-3½" DIAM. PENSTOCKS: PLATE STEEL 10'-2½" DIAM., 136 FT. LONG, SLIDING STEEL HEAD GATES. WATER SUPPLY: INTAKE CANAL ABOUT ONE QUARTER MILE ABOVE FALLS, 282 FT TO 546 FT WIDE, AVERAGE DEPTH 14'-6", LENGTH 300 FT DISCHARGE TUNNEL: HORSESHOE SHAPE, BRICK LINED, 25 FT HIGH BY 18'-10" WIDE, 2165 FT LONG.

FIG. 7.



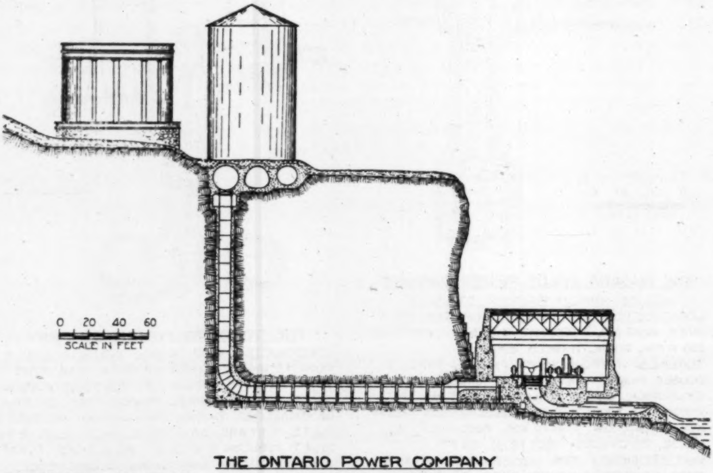
THE TORONTO POWER COMPANY
CONSTRUCTED: 1903 TO 1915. INSTALLATION: 7 15000 H.P. UNITS, 4 13000 H.P. UNITS, 250 R.P.M. TOTAL 11 UNITS 157000 H.P. TURBINES: VERTICAL SHAFT, DOUBLE RUNNER, FRANCIS TYPE, 250 R.P.M. GENERATORS: 7-10000 KVA., 4-8000 KVA., 12000 VOLTS, 3 PHASE, 25 CYCLES. NET HEAD: 135 FT. SHAFT: TUBULAR 2'-8" DIAM. PENSTOCKS: PLATE STEEL 10'-6" DIAM., 145 FT. LONG, SLIDING STEEL HEAD GATE. WATER SUPPLY: INTAKE AND POWER HOUSE ABOUT ONE HALF MILE ABOVE FALLS. DISCHARGE TUNNEL: 2 BRANCH TUNNELS 25 FT HIGH AND 16 FT. TO 28'-9" WIDE, ABOUT 500 FT LONG UNITE AND FORM MAIN HORSESHOE SHAPE TUNNEL 26 FT. HIGH BY 23.5 FT. WIDE, 1935 FT LONG, BRICK LINED.

FIG. 8.

The station was first put into operation in 1906, and was completed by 1915. There are four units of about 13 000 h.p. capacity each, and seven of 15 000 h.p. each, making up a total of 157 000 h.p. for the station. A typical cross-section is shown in Fig. 8. The plant was sold to the Hydro-Electric Power Commission of Ontario in 1918.

Ontario Power Company of Niagara Falls.—The other Company to begin work early in the Twentieth Century was the Ontario Power Company. This Company departed from the types of plant previously built and used pressure conduits about 6 000 ft. long to carry water taken from the river above the Upper Rapids to penstocks connected to water turbines located in a power house in the Gorge just below the Horseshoe Falls.

Power from this plant was first produced in 1905. By 1913, two main conduits, one of plate steel and one of reinforced concrete, each having an internal diameter of 18 ft., and a power house containing fourteen units with a total installed capacity of 186 050 h.p., had been completed. In 1917 this Company was purchased by the Hydro-Electric Power Commission of Ontario. In 1918, as a World War measure, a third conduit, 13½ ft. in diameter, was constructed of wood and used to supply two additional units aggregating 36 000 h.p. capacity. Both of these were destroyed by an accident in 1923, and only one has since been restored. The present installed capacity of the station is 205 350 h.p. A typical cross-section of this plant is shown in Fig. 9.

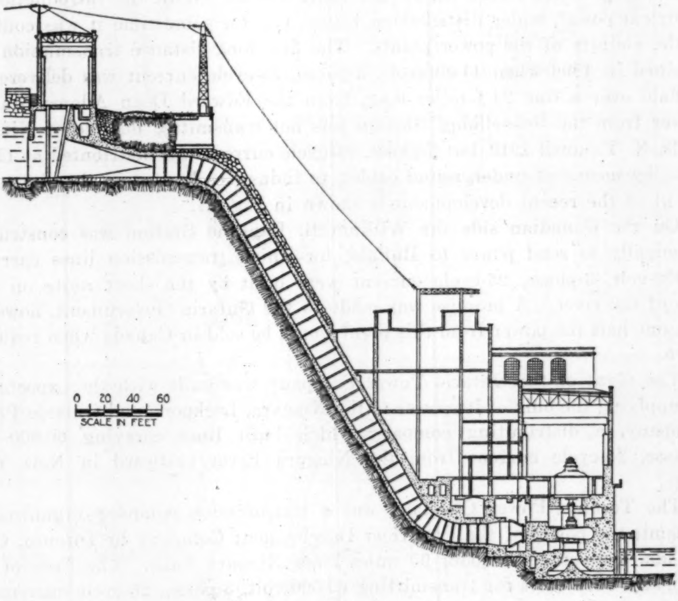


THE ONTARIO POWER COMPANY

CONSTRUCTED: 1902 TO 1923. INSTALLATION: 3-12350 H.P UNITS, 4-13250 H.P UNITS, 4-13500 H.P. UNITS, 3-14000 H.P. UNITS, AND 1-19300 H.P UNIT. 187½ R.P.M. TOTAL 15 UNITS 205350 H.P. TURBINES: HORIZONTAL SHAFT, 2 TURBINES ON SHAFT, FRANCIS TYPE, SCROLL CASE, WICKET GATES. GENERATORS: 3-7500 KVA., 4-8773 KVA., 7-8776 KVA., 1-15000 KVA., 12000 VOLTS, 3 PHASE, 25 CYCLES. NET HEAD: 180 FT. PENSTOCKS: PLATE STEEL VARYING FROM 225 FT TO 307 FT LONG, 14 - 9'-0" DIAM. AND 1 - 10'-6" DIAM., GATE VALVES 12 UNITS AND JOHNSON VALVES 3 UNITS. WATER SUPPLY: INTAKE ABOUT THREE QUARTERS OF A MILE ABOVE FALLS. 3 PIPE LINES, 1 PLATE STEEL, 6300 FT. LONG, 18'-0" DIAM., 1 REINFORCED CONCRETE, 6500 FT. LONG., 18'-0" DIAM., 1 WOOD STAVE, 6700 FT LONG, 13'-6" DIAM. SIMPLE OVERFLOW CHAMBER AT END OF STEEL PIPE LINE AND DIFFERENTIAL SURGE TANKS AT END OF CONCRETE AND WOOD STAVE PIPE LINES.

FIG. 9.

Queenston-Chippawa Development.—The latest power plant built at Niagara Falls was constructed on the Canadian side during the years 1917 to 1921, by the Hydro-Electric Power Commission of Ontario. This plant takes its water from the Niagara River through an intake at the mouth of the Welland River and by reversing the natural flow for a distance of 4½ miles the water is delivered to a canal, 48 ft. wide and about 30 ft. deep, below the water-line. The canal was excavated in earth and solid rock for a distance of 8 miles to a point on the Gorge of the Lower Niagara River about 2 miles above the Village of Queenston, Ont., Canada.



THE HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO
QUEENSTON - CHIPPAWA DEVELOPMENT
CONSTRUCTED: 1917 TO 1925. INSTALLATION: 5-55000 H.P. UNITS AND 4-58000 H.P. UNITS, 187.5 R.P.M., TOTAL 9 UNITS 507000 H.P. TURBINES: VERTICAL SHAFT, SINGLE RUNNER, FRANCIS TYPE, SCROLL CASE. GENERATORS: 5-45000 KVA., 4-55000 KVA., 12000 VOLTS, 3 PHASE, 25 CYCLES: NET HEAD: 305 FT. PENSTOCKS: 437 FT. LONG, 65 FT. OF UPPER HORIZONTAL SECTION SOLID ROCK CONCRETE LINED, 372 FT. PLATE STEEL ENCASED IN CONCRETE, 5 ARE 16'-0" DIAM. FOR ABOUT 2/3 THEIR LENGTH AND THEN REDUCE TO 14'-0" DIAM., AND 4 ARE 16'-0 DIAM. THROUGHOUT, SLIDING STEEL HEAD GATES. JOHNSON VALVES: 14'-0" INLET DIAM., 10'-0" OUTLET DIAM. WATER SUPPLY: INTAKE FROM NIAGARA RIVER AT MOUTH OF WELLAND RIVER, FLOW IN WHICH IS REVERSED FOR A DISTANCE OF 4 1/2 MILES TO CANAL ENTRANCE. CANAL 8 MILES LONG. EXCAVATION IN EARTH AND SOLID ROCK. ROCK SECTION 48 FT. WIDE, 30 FT. DEEP, CONCRETE LINED.

FIG. 10.

The gross head from intake to tail-water is 312 ft. Operation was commenced in December, 1921, and, by 1925, the ninth and last unit was completed. The first five units have capacities of 55 000 h.p. each, and the last four are 58 000 h.p. each, making the total installed capacity 507 000 h.p. A typical cross-section of the generating station is shown in Fig. 10.

TRANSMISSION AND DISTRIBUTION

It will naturally be asked, "Where and how has Niagara power been used?" Of course, in the early stages of development, all the power was used locally in mills by drives connected to the water-wheels. With the introduction of electrical power, wider distribution began, but for some time it was confined to the vicinity of the power plants. The first long-distance transmission was attained in 1896 when 11 000-volt, 3-phase, 25-cycle current was delivered in Buffalo over a line 22.4 miles long, from the Edward Dean Adams Station. Power from the Schoellkopf Station was not transmitted outside of Niagara Falls, N. Y., until 1919, but 3-phase, 25-cycle current was distributed at 12 000 volts, by means of underground cables, to industries located in the city. The extent of the recent development is shown in Fig. 11.

On the Canadian side the William B. Rankine Station was constructed principally to send power to Buffalo, for which transmission lines carrying 22 000-volt, 3-phase, 25-cycle current were built by the short route on that side of the river. A promise was made to the Ontario Government, however, that one-half the power from this plant would be sold in Canada when required there.

The plant of the Ontario Power Company was built with the expectation of supplying the bulk of its power to the Niagara, Lockport, and Ontario Power Company, a distributing company which built lines carrying 60 000-volt, 3-phase, 25-cycle current from the Niagara River eastward in New York State.

The Toronto Power Company was a transmission company organized to transmit the power of the Electrical Development Company to Toronto, Ont., Canada, a distance of about 90 miles from Niagara Falls. The lines of the Company were built for transmitting 60 000-volt, 3-phase, 25-cycle current.

About 1903, a number of municipalities in Western Ontario, noting that no plans had yet been made to send Niagara power to them, and apprehensive lest they might be left in the lurch, petitioned the Government of the Province for permission to form an organization to purchase power from the generating companies at Niagara Falls and transmit it to their municipalities. As a result the Hydro-Electric Power Commission of Ontario was created in 1905, which contracted for the supply of 100 000 h.p. from the Ontario Power Company and built transmission lines to distribute this power throughout the southwestern end of the Province. This system was built for transmitting 110 000-volt, 3-phase, 25-cycle current.

From such beginnings the present distribution (Fig. 11) was gradually developed until now (1928), by purchases and consolidations, the generation and transmission of Niagara power throughout the western part of the State of New York and the southwestern part of the Province of Ontario are largely concentrated in two large systems. One of these serves a population of more than 2 500 000 in the State of New York (and in a small part of the Province of Ontario). The territory involved has an area of 14 500 sq. miles extending to the east as far as Altmar, N. Y., a distance of 200 miles, and to the south, 100 miles to the border of the State of Pennsylvania.

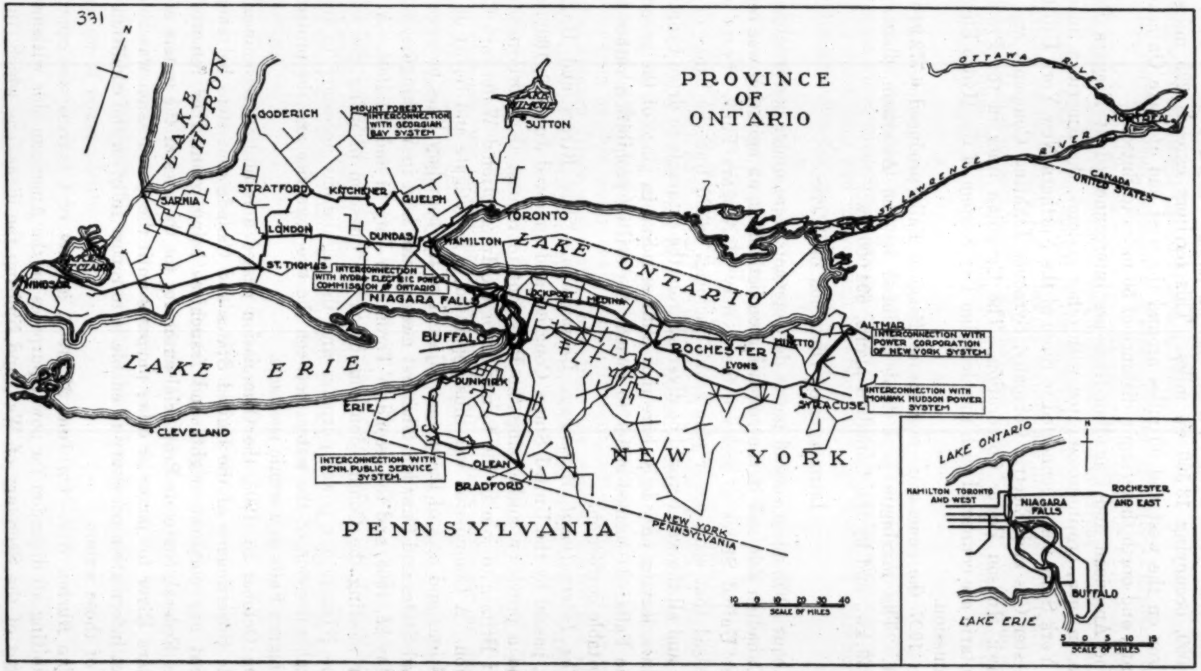

331
N
LAKE HURON
GODERICH
MOUNT FOREST
INTERCONNECTION WITH GEORGIAN BAY SYSTEM
LAKE SIMCOE
SUTTON
PROVINCE OF ONTARIO
OTTAWA RIVER
MONTREAL
ST LAWRENCE RIVER
CANADA
UNITED STATES
SARNIA
STRATFORD
KITCHENER
GUELPH
TORONTO
LONDON
DUNDAS
HAMILTON
LAKE ONTARIO
LAKE ST CLAIR
WINDSOR
ST. THOMAS
INTERCONNECTION WITH HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO
NIAGARA FALLS
LOCKPORT
MEDINA
ALTMAR
INTERCONNECTION WITH POWER CORPORATION OF NEW YORK SYSTEM
BUFFALO
ROCHESTER
MINETTO
LYONS
LAKE ERIE
DUNKIRK
INTERCONNECTION WITH MOHAWK HUDSON POWER SYSTEM
SYRACUSE
ERIE
CLEVELAND
INTERCONNECTION WITH PENN. PUBLIC SERVICE SYSTEM
OLEAN
BRADFORD
NEW YORK
PENNSYLVANIA
NEW YORK
PENNSYLVANIA
10 0 10 20 30 40
SCALE OF MILES
LAKE ONTARIO
N
HAMILTON TORONTO AND WEST
NIAGARA FALLS
ROCHESTER AND EAST
BUFFALO
LAKE ERIE
5 0 5 10 15
SCALE OF MILES

FIG. 11.—MAP OF TERRITORIES SERVED BY NIAGARA POWER.

In the Province of Ontario, the other system serves a population of about 1 500 000, occupying 13 500 sq. miles. This territory extends 225 miles to Windsor on the west and 90 miles around the west end of Lake Ontario to Toronto, and north therefrom a distance of 50 miles to Sutton.

The American and Canadian lines are interconnected at Niagara Falls. These, in turn, are interconnected with other systems, the American lines in New York State being connected to those of the Northern New York Utilities, Rochester Gas and Electric Company, Syracuse Lighting Company, and the Mohawk Hudson Power Corporation. The Canadian lines in the Province of Ontario are connected with the Georgian Bay System of the Hydrc-Electric Commission.

In 1927 the generating stations at Niagara Falls produced 6 772 944 200 kw-hr. The maximum peak load produced by the American plant was 365 659 kw., and by the Canadian plants, 694 000 kw.

LIMITATION OF DIVERSION OF WATER

About 1906, when several power developments were under construction on the Canadian side and in contemplation on both sides, an agitation was begun in the United States to preserve the scenery of Niagara Falls. Fears were expressed that soon power plants and factories would line the banks of the river and all the water would be diverted from the cataracts to drive hydraulic turbines, leaving only ugly, bare rock cliffs standing in place of the grandeur of the Falls. It seemed easier to remedy the evil by prohibition rather than by suitable correction.

The International Waterways Commission.—"The Rivers and Harbors Act", passed by the United States Congress and approved June 13, 1902, contained a provision authorizing the President to request the Government of Great Britain to join in the formation of the International Waterways Commission. A Commission was thus appointed to investigate and report on the conditions and uses of the waters adjacent to the boundary lines between the United States and Canada. The first meeting was held in Washington, D. C., on May 25, 1905, and the second in Toronto on July 14 and 15, 1905. At the latter meeting the Commission considered the question of using the waters of the Niagara for power purposes and the regulations necessary to insure equitable division of the waters between the two countries and the protection of Niagara Falls as a scenic spectacle.

On October 28, 1905, the Commission passed a resolution recommending to the Governments of the United States and Canada that steps be taken to prevent any corporate rights and franchises being granted or renewed by either Federal, State, or Provincial authority for the uses of the waters of the Niagara River for power or other purposes until the Commission was able to collect information necessary to enable it to report fully on the condition and uses of those waters.

The Burton Bill.—On June 29, 1906, an Act of Congress was approved prohibiting all diversion for power purposes on the American side without the consent of the Secretary of War, and limiting the diversions which, in any event, could be permitted to 15 000 cu. ft. per sec. This bill remained in force for three years. It was renewed for two years from June 29, 1909, and after two other extensions it lapsed March 4, 1913.

The International Boundary Waters Treaty of 1910.—Pursuant to the terms of the Burton Act and following the investigations of the Waterways Commission, a treaty between the United States and Great Britain was ratified in 1910 and is now known as the Boundary Waters Treaty.

Among its provisions, the following clauses are of special interest:

"The High Contracting Parties shall have each on its own side of the boundary equal and similar rights in the use of the waters hereinbefore defined as boundary waters."

* * * * * *

"The requirements for an equal division may in the discretion of the Commission, be suspended in cases of temporary diversions along boundary waters at points where such equal division cannot be made advantageously on account of local conditions and where such diversion does not diminish elsewhere the amount available for use on the other side."

* * * * * *

"The High Contracting Parties agree that it is expedient to limit the diversion of waters from the Niagara River so that the level of Lake Erie and the flow of the stream shall not be appreciably affected. It is the desire of both Parties to accomplish this object with the least possible injury to investments which have already been made in the construction of power plants on the United States side of the river under grants of authority from the State of New York, and on the Canadian side of the river under licenses authorized by the Dominion of Canada and the Province of Ontario.

"So long as this treaty shall remain in force no diversion of the waters of the Niagara River above the Falls from the natural course and stream thereof shall be permitted except for the purposes and to the extent hereinafter provided.

"The United States may authorize and permit the diversion within the State of New York of the waters of said river above the Falls of Niagara, for power purposes, not exceeding in the aggregate a daily diversion at the rate of twenty thousand cubic feet of water per second.

"The United Kingdom, by the Dominion of Canada, or the Province of Ontario, may authorize and permit the diversion within the Province of Ontario of the waters of said river above the Falls of Niagara, for power purposes, not exceeding in the aggregate a daily diversion at the rate of thirty-six thousand cubic feet of water per second.

"The prohibition of this article shall not apply to the diversion of water for sanitary or domestic purposes, or for the service of canals for the purpose of navigation."

In view of the first clause quoted, it is interesting to inquire what possible reasons led to the inequality in the diversions permitted at Niagara Falls. The treaty itself gives no specific reason for allowing 36 000 cu. ft. per sec. to Canada and 20 000 cu. ft. per sec. to the United States, beyond stating that it is the desire of both parties to accomplish the object of the treaty with the least possible injury to investments which had already been made in the construction of power plants.

It is probable that there were at least four factors which led to the unequal division:

1.—The desire to impose limitations on the total diversions without injury to investments already made.

2.—The equalizing effect of generating power on the Canadian side and transmitting it to the United States.

3.—The position of two of the plants on the Canadian side, with intakes below the crest of the upper cascades and, therefore, not affecting the American Falls.

4.—The works constructed at Chicago, Ill., for the diversion of water from the Great Lakes.

The limitation of diversion of water imposed by the treaty restricts the aggregate output of the power plants at Niagara Falls to an amount considerably below their combined installed capacities. It is for this reason that the Edward Dean Adams Plant is held in reserve as previously stated.

Table 1 shows power statistics for the various plants when operated in accordance with the provisions of the treaty.

TABLE 1.—LIMITS OF OPERATION, NIAGARA POWER PLANTS.

Plant.	Installed capacity, in horse-power.	PRESENT USE UNDER TREATY LIMITATIONS.	
		Average horse-power.	Peak horse-power.
Schoellkopf	452 500	417 000	488 000
Edward Dean Adams	105 000	0	0
William B. Rankine	121 000	100 000	100 000
International Railway Company	3 600	....	....
Ontario Power Company	205 350	130 000	180 000
Toronto Power Company	157 000	20 000	100 000
Queenston-Chippawa	507 000	470 000	520 000
Total	1 551 450	1 137 000	1 388 000

EFFECT OF POWER DEVELOPMENT ON LAKE ERIE AND THE NIAGARA RIVER

Relation Between Lake Erie Stage and Niagara Discharge.—The equation relating the discharge of the Niagara River to the stage of Lake Erie has been determined by the engineers of the United States Government as follows:

$$Q = 3\,904\,(H - 558.37)^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which,

Q = discharge of the river, in cubic feet per second; and,

H = elevation above mean sea level of the surface of the water in the lake, Buffalo Gauge.

The mean value of H for the period, 1860 to 1927, inclusive, was 572.50.

Considering the relation of Equation (1) roughly, it may be stated that, when the stage of Lake Erie has risen 1 ft. above this mean level, there will be an increase of about 22 400 cu. ft. per sec. in the rate of discharge of the Niagara River; and, when the stage of the lake has fallen 1 ft. below its mean level, there will be a corresponding reduction in discharge of about 21 600 cu. ft. per sec.

Variations in Lake Levels and Discharge.—There are three kinds of variations in lake level and discharge of the Niagara River:

(1) Seasonal variations every year.
(2) Cycles of change due to variable supply from precipitation and run-off.
(3) Daily fluctuations on account of the changes in atmospheric pressure and in the direction and velocity of winds.

A study of the records of the monthly mean lake levels and discharges of the Niagara River will show that the normal seasonal variation in lake level is about 1.5 ft., and that the normal seasonal variation in discharge of the Niagara River is about 31 900 cu. ft. per sec.

In addition to the seasonal variations there are periodic fluctuations in lake level and corresponding variations in river discharge extending over a period varying from seven to eleven years, the effect of which is to make the difference between the maximum mean monthly level of Lake Erie and its minimum mean monthly level about 5.0 ft. These variations correspond to a difference between the maximum and the minimum discharges of the Niagara River of about 107 900 cu. ft. per sec.

Changes in the direction and velocity of winds on Lake Erie and in atmospheric pressure produce greater fluctuations, than the foregoing and independent of them, but only for periods of short duration. The difference between the maximum and the minimum level of the lake from this cause is about 13.0 ft. Table 2 shows the ranges in level and the corresponding discharges for the period, 1860 to 1927, inclusive.

TABLE 2.—VARIATIONS OF LEVEL IN LAKE ERIE AT BUFFALO, N. Y., AND OF DISCHARGE OF THE NIAGARA RIVER.

	Elevation, in feet.	Discharge, in cubic feet per second.
Average monthly mean (1860–1927 inclusive)	572.50	207 400
Average maximum monthly mean	573.18	222 500
Average minimum monthly mean	571.73	190 600
Maximum monthly mean (June, 1876)	574.68	257 200
Minimum monthly mean (February, 1926)	569.72	149 300
Maximum lake level due to wind	580.28	....
Minimum lake level due to wind	567.38	....

Effect of Diversions.—It is claimed that the navigable depths of Lake Erie have been rendered insufficient from various causes, such as:

(1) Deficient precipitation and run-off.
(2) Diversions of water from the Great Lakes through:
    a.—Chicago Drainage Canal
    b.—Welland Canal
    c.—Black Rock Ship Canal
    d.—New York State Barge Canal
    e.—Power plants at Niagara Falls.

Prior to 1927, the most authoritative estimate of the amount of the lowering at mean stage caused by all these diversions was 0.76 ft.[4] Of this total, the amount of lowering caused by diversions for power purposes at Niagara Falls was considered to be 0.1 ft., at least one-half of which had been compensated for by changes in the bed of the river. The amount of this lowering has been placed by other estimates as high as 3½ in.

---

[4] Report on the Diversion of Water from the Great Lakes and Niagara River, by J. G. Warren, M. Am. Soc. C. E., p. 375.

Actually, however, measurements made in the Niagara River during the summer of 1925, under the direction of the District Engineer Office, at Buffalo, indicate that there has been no appreciable change in the relation between the discharge of the river and the stage of Lake Erie since 1900 when measurements were made by the U. S. Lake Survey Office under the direction of Francis C. Shenehon, M. Am. Soc. C. E. Such differences as were found between the measurements of 1925 and 1900 were in the opposite direction, indicating that the level of Lake Erie for a given discharge of the Niagara River had become slightly higher than in 1900. This is due probably to compensatory changes in the shore line and bed of the river, which have offset any effects that otherwise might have been caused by diversion.

Back-water studies of the flow of the Niagara River indicate that it would require a lowering of approximately 2.0 ft. in the river level at Port Day to cause a decrease in the level of Lake Erie by as much as 3.5 in. No such lowering at Port Day has occurred.

Proposed Regulation.—It has been proposed from time to time to hold Lake Erie, as well as the other Great Lakes, as nearly as possible at a uniform elevation, in order that the navigable depth will not be lessened during the low-water season each year, particularly in years of low levels following periods of deficient natural supply. For this purpose various means have been suggested, such, for instance, as a regulating dam with gates and sluices at the outlet of Lake Erie, or across the Niagara River between Port Day and Chippawa, Ont., Canada. In operating such a dam it is proposed to raise or lower the gates so as to control the outflow from the lake, allowing more water to flow out when the lake is rising and less when it is falling. It is not usually expected by this means to hold it at a uniform level, but merely to restrict the range of variation to something less than when it is left free under natural conditions.

Evidently the monthly mean level of the lake cannot be kept more nearly uniform without at the same time causing more variation in the monthly mean flow of the Niagara River, assuming that the supply to the lake will vary from natural causes as in the past. In periods of deficient supply a part of the flow of the river comes from the water stored in the lake, thereby causing lake levels to fall; and in periods of excess supply the water cannot flow through the river as fast as it enters the lake, thus causing lake levels to rise.

If Lake Erie is to be regulated in such a way that the difference between the maximum and the minimum mean monthly water level will be less than in the past, the effective storage capacity of the lake will be reduced. Then the difference between the volumes of water discharged in the Niagara River at rates higher and lower than the mean must necessarily be greater than in the past. Conversely, if the flow of the river is to be made more nearly uniform, a greater storage capacity will be required; and, consequently, the difference between the maximum and the minimum level of Lake Erie must be greater than it has been in the past. If it is desired to increase the depth of Lake Erie for navigation purposes and, at the same time, to obtain greater regularity of flow in the Niagara River, it would be necessary to provide regulating works which will have the effect of raising the mean level of the lake to a higher plane. The necessary fluctuations in lake level would then occur above and below this plane to the extent necessary to maintain the greatest possible degree of regularity in the flow of the river.

The area of Lake Erie is approximately 10 000 sq. miles. When its level is lowered 1 ft. below the mean, a volume of water is supplied to the Niagara River sufficient to maintain its entire average flow for about sixteen days. If the flow of the river were completely shut off for one day, the use of the water for power and scenic purposes would cease, but the level of the lake would be raised only about 0.75 in. It is thus apparent that obstructions which tend to interfere temporarily with the flow of the river are of much greater significance in respect to variation in flow than to water level in the lake.

Proposed Regulation and Compensation Works.—Regulating works and compensating works for Lake Erie have been proposed from time to time since about 1900. The object of regulating works is to control the outflow so as to increase the navigable depths during otherwise naturally low-water periods.

Compensating works are different in that they are designed only to raise the mean level of the lake without attempting to control the natural variations. Fluctuations will occur above and below a certain plane higher than that existing before the works were constructed; but the depths at all stages will be greater.

Location of Proposed Regulating Works.—It has been proposed that such works be located either at the entrance to the Niagara River, near the outlet of Lake Erie, or in the river, between Port Day and Chippawa. If the value of lake regulation is sufficient to offset the economic loss of the present natural conservation of water supply for power purposes in the Niagara and St. Lawrence Rivers, the former location is preferable.

A regulating dam between Port Day and Chippawa just above the Falls, would offer serious difficulties from the standpoint of power development. No structure must be placed in the Niagara River which will endanger the supply of water to the power plants by interfering with the passage of the large volumes of ice each winter. The height to which the water would have to be raised at Port Day to produce an appreciable effect on the levels of Lake Erie would be so great that important additions would have to be made to the power plants.

Lands lying along the river above Niagara Falls and along the tributary creeks, would have to be protected and drained. The sewage of La Salle, Tonawanda, and North Tonawanda, N. Y., would have to be cared for. A guard-lock would have to be constructed at the entrance to the Barge Canal at Tonawanda; and it is probable that the Buffalo Breakwater and the dike forming the Black Rock Ship Canal would have to be modified.

Referring now to the economic loss of power incurred by regulating Lake Erie so that its variation in level will be less than in the past, it will be assumed that the usable flow of the river will be some function of the "50% time flow" referred to by the Advisory Committee of the International Electrotechnical Commission of Rating of Rivers. It will be noted that the 50% time flow of the Niagara River may be utilized with an average load factor as high as 96 per cent.

One proposed rule of regulation may be applied to the measured discharge of the Niagara River during the period 1860 to 1924, inclusive, on the assumption that the minimum flow of the Niagara River would not be permitted to be less than 180 000 cu. ft. per sec. Then to obtain a regulation of mean monthly levels of Lake Erie within a range of 1.5 ft. and to provide for a diversion of 13 000 cu. ft. per sec. from and above the lake for navigation and other purposes, it would be necessary to maintain the regulated minimum flow of 180 000 cu. ft. per sec. approximately 68% of the time.

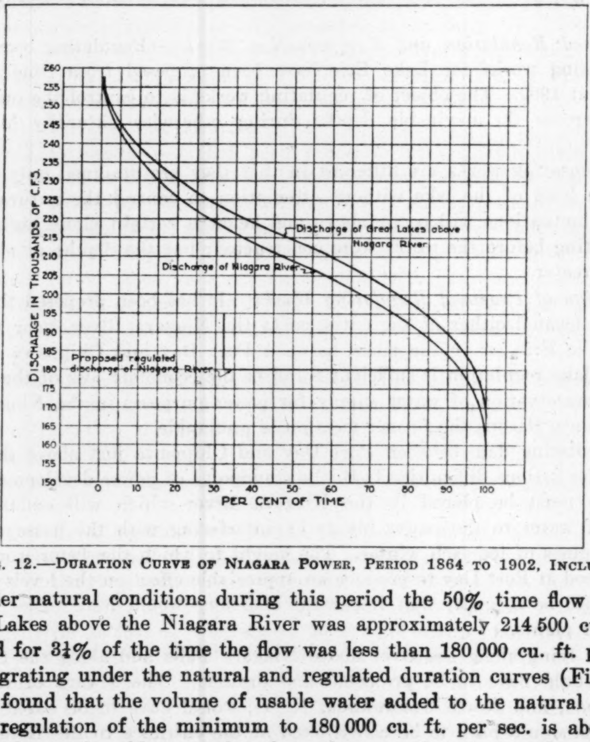


FIG. 12.—DURATION CURVE OF NIAGARA POWER, PERIOD 1864 TO 1902, INCLUSIVE.

Under natural conditions during this period the 50% time flow from the Great Lakes above the Niagara River was approximately 214 500 cu. ft. per sec. and for 3½% of the time the flow was less than 180 000 cu. ft. per sec.

Integrating under the natural and regulated duration curves (Fig. 12), it will be found that the volume of usable water added to the natural low flows by the regulation of the minimum to 180 000 cu. ft. per sec. is about 1.8% of the volume of usable water lost from the natural 50% time flow. The net loss to power development on the Niagara River is, therefore, nearly 2 000 000 000 kw-hr. per year. To this would have to be added also the loss of power on the St. Lawrence River.

The value of this power must be considered as a charge against the benefits to be derived from regulated navigable depths and from the use of water diverted for other purposes. This conclusion is not altered by reason of the fact that part of the flow of the Niagara River is required for scenic purposes, because such water, which has been fixed by some authorities at 80 000 cu. ft. per sec., is considered to be the first claim on the river, and what remains is to be available for power purposes. The difference between the usable quantities indicated is not altered, therefore, by any such fixed allowance for scenic purposes.

ECONOMIC BENEFITS OF NIAGARA POWER

The question of the value of Niagara power to the districts which it serves is especially important because so much is being said about the remarkable and undeniable accomplishments in the field of steam-electric generation. It would be a mistake, however, to assume that recent great improvements in the economy of steam generation, forming as they do one of the brightest chapters in the record of the Engineering Profession, have yet reached the point, or are indeed likely to reach that point, where all water power may be thrown into the discard.

Comparisons of the cost of electric service in widely separated areas of the Continent, while of considerable interest, are perhaps of doubtful significance because charges for such service are so largely dependent on economic conditions in the localities where the service is rendered. Rates for electric light and power depend primarily on the capital costs of the generating, transmitting, and distributing systems, and on the number of kilowatt-hours which can be sold per dollar of capital invested. The former, in turn, depend on the physical conditions affecting the construction of the systems and on the costs of labor and materials, and the value of land. The kilowatt-hours, on the other hand, depend on the density of population and the value to the user of the service rendered. Taxes and the amounts reserved for plant renewals are important items of annual expense in addition to labor cost and interest on the invested capital.

There is a tendency to assume in public discussions of electric rates, that electricity as a commodity differs from nearly every other one in the world, in that it can be sold for the same price no matter where or under what conditions it is produced. It is also frequently assumed that electric service is a monopoly requiring something more than the ordinary economic laws to regulate its price.

Protection of the public interest by regulatory commissions does not alter the fact that in many ways the electrical business is highly competitive. Electricity competes with gas, oil, and other fuels in the same district. One district competes with another for the growth of business, and some industries largely dependent on electricity compete with like industries over the entire world. More especially, perhaps, electricity competes with all other commodities, such as motor cars, radios, etc., for its share of the consumer's dollar. Such competition effectively tends to regulate prices of electric service to their minimum economic level.

TABLE 3.—Niagara Power—Cost and Capacity, Including Auxiliary Plants, 1926.

Territory.	Fixed electrical capital.	Installed capacity, in kilowatts.	Capital per kilowatt.
Western New York	$205 932 420	770 989	$267
Southwestern Ontario	$251 938 759	814 826	$309

The data in Tables 3 and 4, taken mainly from published financial reports, are presented not to indicate that rates for electricity in some other district should be different from what they are, but merely to show how the large territories served by Niagara power have benefited by the utilization of a small part of the energy made available by this great gift of Nature. Later, it will be indicated how further developments may be made without materially sacrificing those æsthetic benefits which are to be derived by the contemplation of the scenery of the Falls.

TABLE 4.—Niagara Power—Revenues, 1926.

Territory.	Electrical operating revenue from sale of electrical energy.	Electrical operating revenue from sale of electrical energy (taxes deducted).	Total kilowatt-hours generated or purchased.	Revenue, in Mills per Kilowatt-Hour.	
				Taxes included.	Taxes deducted.
Western New York	$41 558 864	$36 531 624	4 495 959 612	9·24	8·13
Southwestern Ontario	26 093 911	25 804 658	2 821 791 607	9·25	9.14

The average revenue received per kilowatt-hour from the sale of power in other territories less favorably situated and served largely by steam, is from two and one-half to three times as great. These statistics prove conclusively, therefore, that the economic benefits of Niagara power have been passed on to the users. They disprove the assumption, frequently made, that the difference between the cost of water power and steam power is collected from the consumer and retained wholly for the benefit of the shareholders of the companies undertaking the development.

ULTIMATE DEVELOPMENT OF NIAGARA FALLS

Preservation of their scenic grandeur is the most important factor involved in the ultimate development of the Falls of Niagara. The discussion of this subject would be quite simple if it could be based on Lord Kelvin's prophecy, which, on the occasion of his visit to Niagara Falls in 1897, he expressed in these words:

"I look forward to the time when the whole water from Lake Erie will find its way to the lower level of Lake Ontario through machinery, doing more good for the world than even that great benefit which we now possess in contemplation of the splendid scene which we have before us in the waterfall of Niagara. I wish I could live to see this grand development. I do not hope that our children's children will ever see the Niagara cataract."

Under such conditions scenery would become a negligible factor, and power can be imagined as enthroned in two huge generating stations, extending half a mile along each side of the river at the foot of the Lower Rapids near Lewiston and Queenston, Ont. Giant generating units would produce 7 000 000 h. p. "on peak" and an average of about 6 000 000 h. p. daily throughout the year.

Such was the vision of Lord Kelvin thirty years ago. To-day, it is doubtful whether any one would venture to make such a prediction. On the contrary, for the past twenty years attention has been directed not only to the possibility of diverting more water for power purposes, but to showing how to preserve the scenery of the Falls and Rapids by providing works to remedy the effects of diversion and of the continued rapid recession of the crest of the Horseshoe Falls.

Major Keller.—Commencing with the report of Maj. Charles Keller, U. S. A. (now Colonel, U. S. A. (*Retired*)), M. Am. Soc. C. E., in 1908 on the preservation of Niagara Falls, a few quotations will be given. Major Keller states:

> "In my opinion, the damage already done, and that which may be anticipated from further diversions and from the impending fall in the level of Lake Erie, may be largely, if not entirely, remedied by a submerged dam placed in the bed of the river immediately above the Horseshoe Falls."

International Waterways Commission.—About the same time the International Waterways Commission, in its joint report dated 1908, made the following recommendation with reference to diversions around the Lower Rapids:

> "It is our opinion that about 40 000 cubic feet per second can be diverted without perceptible injury to the rapids, * * *."

John L. Harper.—Following these two recommendations, the late John L. Harper, M. Am. Soc. C. E., in 1918, published a pamphlet directing attention to the need for the preservation of scenic effects, stating:

> "* * * it should be the policy of those controlling the falls at Niagara to have constructed in the bed of the river, above the Horseshoe Falls, invisible current deflectors which would make impossible the gathering of the whole river into a deep narrow gorge, and would again deflect its waters over the sides and heels of a re-established horseshoe * * * causing the whole contour of the fall to wear uniformly * * *."

Colonel J. G. Warren.—One year afterward, the well-known Warren report of Col. J. G. Warren, U. S. A. (*Retired*), M. Am. Soc. C. E., then Division Engineer at Buffalo, contained the following:

> "If the remedial works * * * are provided, it is believed a total diversion of 80 000 cubic feet per second may be made around the Falls, and 40 000 around the Whirlpool and Lower Rapids without injury to the scenic beauty, and without endangering the ice-discharging capacity of the Falls or Rapids, these diversions to be divided equally between Canada and the United States."

Board of Engineers for Rivers and Harbors.—This report was then submitted to the Board of Engineers for Rivers and Harbors, headed by the late Maj. Gen. H. Taylor, U. S. A., (*Retired*), M. Am. Soc. C. E., which, in a report dated August 24, 1920, approved the recommendations of the Division Engineer as to the necessity for remedial works, as follows:

"So far as concerns the preservation of scenic beauty, the most important measure suggested by the Division Engineer is the construction of a weir and certain rock excavation for the uniform distribution of flow over the Horseshoe, * * *. To preserve the flow of the American Fall he recommends a rough submerged weir between the head of Goat Island and the Canadian side, part of which has already been made by dumping dredge spoil.

"We are in accord with these recommendations of the Division Engineer as to the advantage of distributing as nearly uniformly as possible the water flowing over the Horseshoe Fall and as to the general method by which this can be accomplished."

* * * * *

"No definite limit should be set upon the critical discharge over the Falls and the amount of water permitted to be diverted, other than to state that the remedial works should be designed so as to afford the maximum attainable scenic beauty, in our opinion corresponding to an ultimate diversion of 100 000 to 110 000 cubic feet per second."

Special International Niagara Board.—Finally, in 1926, the Governments of the United States and Canada appointed a Commission, known as the Special International Niagara Board, to study and report on the problem. Pending the preparation of its final report, the Board issued an interim report in December, 1927, which contains interesting recommendations covering the construction of initial remedial works at an estimated cost of $1 750 000, for the preservation of the scenic values of the Falls.

This report led to an agreement between the United States and Canada covering the construction of the remedial works recommended by the Board. The agreement was embodied in a Convention and Protocol signed in Ottawa, Ont., Canada, January 2, 1929, ratified by the Parliament of Canada, May 20, 1929, and, at present (June, 1929), is awaiting ratification by the Senate of the United States.

Scenery and Power.—Evidently, the ultimate development of the Falls of Niagara, for many years at least, will represent a compromise between the demands of scenery and of power. Lest it should be inferred that these are always opposed to one another, it may be noted that the requirements of power actually may be made to serve the needs of scenery. Furthermore, economic pressure surely will demand the same efficiency in the use of water for scenery as for power.

The maximum diversions officially recommended to date, however, are only 100 000 cu. ft. per sec. around the Falls and 40 000 cu. ft. per sec. around the Lower Rapids, provided suitable works are constructed for the preservation of the scenery and the regulation of the river.

Existing Plants.—The distribution of power and use of water in the existing plants is shown on Fig. 1. No attempt has been made to account for small quantities of treaty water, such as those used through the New York State Barge Canal and in the plant of the International Railway in Canada. Neither will the subsequent discussion be complicated by reference to the division of water between the United States and Canada. The principle of equality in respect of the use of boundary waters has already been agreed upon and ultimately the benefits of Niagara diversion will be divided equally between the two countries.

The notes on Fig. 1 show that, with the present allowed diversion of 56 000 cu. ft. per sec., a daily average output of 1 137 000 h.p. and a peak of about 1 388 000 h.p. are being produced. A ratio of 1 : 1.2 between average and peak for the plants on the American side has been fixed by the Federal Power Commission. No ratio has yet been fixed on the Canadian side.

An average of about 16 000 cu. ft. per sec. are being diverted around both the Falls and the Rapids. The remaining 40 000 cu. ft. per sec. are being diverted around the Falls only.

Proposed New Plants.—If new plants using a 312-ft. head are built to utilize the 24 000 cu. ft. per sec. remaining out of the 40 000 now recommended for diversion around the Falls and Rapids, the present plants (1928) are sufficient to utilize the remaining 60 000 cu. ft. per sec. around the Falls. Under these conditions, the ultimate development would give about 2 100 000 h.p. for the daily average and 2 520 000 h.p. on peak. By scrapping the most inefficient plants (marked 2, 3, and 5 on Fig. 1) and building new plants of modern type to utilize the 220-ft. head from above the Upper Rapids to the Maid-of-the-Mist pool, the maximum development under present recommendations would be increased to a daily average of 2 500 000 h.p., and, with the Federal Power Commission's ratio, to about 3 000 000 h.p. on peak. Thus, recommendations have already been made which would permit a total development of power twice as great as that provided by the existing plants.

It is probable, however, that more than 40 000 cu. ft. per sec. may be diverted around the Lower Rapids without serious impairment of their scenic beauty or their ice-discharging capacity. If the total quantity recommended for diversion around the Falls were permitted to be diverted also around the Lower Rapids, the total development would rise to 3 000 000 h.p. average and 3 600 000 h.p. peak. From that point onward the ultimate development of Niagara can be estimated only by determining the minimum quantity of water required to preserve a satisfying scenic spectacle.

When the limits of diversion consistent with the preservation of a satisfactory scenic spectacle are reached, it is probable that it will become necessary to find means to utilize as much as possible of the flow of the river during the night hours when scenic effects may be disregarded. This may be done by installing additional capacity in industrial plants to be supplied with power only during the night hours, or by substituting hydro-electric power for steam power to save fuel that would otherwise be burnt at night, or by storing the water for use at hours of peak load during the day. The greater value of peak power, which results from the instinct that leads most people to prefer to work during the day and sleep at night, may tend to offset the cost of storage. Such expedients as these will probably form part of the ultimate development of Niagara Falls.

It is most important to remember, however, that at this moment it is not at all necessary to decide how far Niagara power development, ultimately, may be carried. The preservation of a satisfactory spectacle may be insured by a step-by-step development of power and scenery together. A diversion of 20 000 cu. ft. per sec. is sufficient water for the first step; this will provide for power development on a sound economic basis. As each development is made, its effect on the scenery may be noted, and if necessary remedial works may be provided. It is beyond question that the logical step immediately required in the development of Niagara power is the construction of the remedial works recommended by the Board and a new plant on the American side under a head of 312 ft. to utilize initially not less than 10 000 cu. ft. per sec. and produce not less than 300 000 h.p.

BIBLIOGRAPHY

Many references to Niagara plants and to the important problems involved are found in engineering literature. For convenience a number of the more important papers are listed here, under appropriate subject headings.

GENERAL

Electrical Transmission from Niagara, by Benjamin Rhodes. *Transactions,* Am. Soc. C. E., Vol. XIV (1885), p. 205.

Hydro-Electric Power in Canada, by Cecil B. Smith. *Transactions,* Am. Soc. C. E., Vol. LXV (1909), p. 154.

Power Development at Niagara Falls, by Morris Cohn, Jr., and John L. Harper. *Electrical World,* Vol. 75, No. 12, March 20, 1920.

Hydro-Electric Development at Niagara Falls, by John L. Harper and J. Allen Johnson. *Journal,* A. I. E. E., Vol. 40, No. 7, July, 1921.

SCHOELLKOPF STATION No. 2

Description of Plant. *Electrical World,* January 14, 1899.

Niagara Power in the Gorge—I. *Electrical World and Engineer,* Vol. 46, No. 21, November, 1905.

SCHOELLKOPF STATION No. 3-A

Niagara Power in the Gorge—II. *Electrical World and Engineer,* Vol. 46, No. 22, November, 1905.

Description of Plant, Textbook, "Hydraulic Turbines," by Gelpke and Van Cleve.

STATION No. 3-B

Features of a Mammoth Waterwheel Generator. *Electrical World,* Vol. 73, No. 22, May 31, 1919.

A 32 500 Kv-a. Waterwheel Generator, by R. B. Williamson. *Electrical World,* Vol. 74, No. 9, August 31, 1919.

Features of Design in Large Hydraulic Turbines. *General Electric Review,* Vol. 22, No. 11, November, 1919.

World's Largest Hydro-Electric Units, by F. A. Annett. *Power,* Vol. 52, No. 11, September 14, 1920.

Design of World's Most Powerful Hydraulic Turbines, by Lewis F. Moody. *Canadian Engineer,* September 16, 1920.

Construction Features of The Niagara Falls Power Company's Plant, by O. D. Dales. *Canadian Engineer,* September 16, 1920.

Concrete Slab 11 Ft. Thick, 45 Ft. Span Carries Each Machine, by Louis S. Bernstein. *Canadian Engineer,* September 16, 1920.

Engineering and Surveying for Penstock Excavation, by G. W. Hewitt. *Canadian Engineer,* September 16, 1920.

Niagara Development Breaks Efficiency Records. *Canadian Engineer,* September 16, 1920.

Hydraulic Design, Test Results and Methods of Testing, by Norman R. Gibson. *Canadian Engineer,* September 16, 1920.

Niagara Falls 100 000 H.P. Development, by John L. Harper. *Electrical World,* Vol. 76, No. 12, September 18, 1920.

General Engineering Problems Involved in the Development, by George R. Shepard. *Electrical World,* Vol. 76, No. 12, September 18, 1920.

Hydraulic Design and Efficiency of Units and Plant, by Norman R. Gibson. *Electrical World,* Vol. 76, No. 12, September 18, 1920.

Effect of Loading on Construction of Substructure, by L. S. Bernstein. *Electrical World,* Vol. 76, No. 12, September 18, 1920.

Developing an Additional 100 000 H.P. at Niagara. *Engineering News-Record,* Vol. 85, No. 13, September 23, 1920.

How the Needs of War Developed Co-Operation at Niagara, by John L. Harper. *Engineering News-Record,* Vol. 85, No. 13, September 23, 1920.

Structural and Equipment Features of New Niagara Plant, by O. D. Dales. *Engineering News-Record,* Vol. 85, No. 13, September 23, 1920.

Details of Turbine Settings, by L. S. Bernstein. *Engineering News-Record,* Vol. 85, No. 13, September 23, 1920.

Design of 37 500 H.P. Turbine at Niagara. *Engineering News-Record,* Vol. 85, No. 14, September 30, 1920.

Design of the Allis-Chalmers Unit, by W. M. White. *Engineering News-Record*, Vol. 85, No. 14, September 30, 1920.

Details of the I. P. Morris Unit, by Lewis F. Moody. *Engineering News-Record*, Vol. 85, No. 14, September 30, 1920.

Hydraulic Problems of the Design, by George R. Shepard and Norman R. Gibson. *Engineering News-Record*, Vol. 85, No. 14, September 30, 1920.

Features of 37 500 H.P. Water Turbine. *Electrical World*, Vol. 76, No. 14, October 2, 1920.

Waterwheel and Generator Considered as Homogeneous Unit, by W. M. White. *Electrical World*, Vol. 76, No. 14, October 2, 1920.

Two Waterwheels for Generators of Different Make, by Lewis F. Moody. *Electrical World*, Vol. 76, No. 14, October 2, 1920.

Building the American Niagara Power Extension. *Engineering News-Record*, Vol. 85, No. 15, October 7, 1920.

Enlarging Channels and Erecting the Station, by O. D. Dales. *Engineering News-Record*, Vol. 85, No. 15, October 7, 1920.

Tunneling and Lining the Penstocks, by G. W. Hewitt. *Engineering News-Record*, Vol. 85, No. 15, October 7, 1920.

Factors Determining the Design of Generators. *Electrical World*, Vol. 76, No. 16, October 16, 1920.

Reliability Is Keynote of Generator, by F. D. Newbury. *Electrical World*, Vol. 76, No. 16, October 16, 1920.

Operating Experiences Prove Correctness of Design of 32 500 Kv-a. Generator, by R. B. Williamson. *Electrical World*, Vol. 76, No. 16, October 16, 1920.

Design of 32 500 Kv-a. Generator Is Conservative, by W. J. Foster. *Electrical World*, Vol. 76, No. 16, October 16, 1920.

Unit Idea in 100 000 H.P. Electrical Layout, by J. Allen Johnson. *Electrical World*, Vol. 76, No. 17, October 23, 1920.

600 000 H.P. Line Delivers Niagara Power, by J. Allen Johnson. *Electrical World*, Vol. 76, No. 18, October 30, 1920.

Clearing House for Five Stations, by J. Allen Johnson. *Electrical World*, Vol. 76, No. 20, November 13, 1920.

New Method of Water Measurement in Efficiency Tests of 37 500 H.P. Turbines, by Norman R. Gibson. *Power*, Vol. 53, No. 12, March 22, 1921.

Station No. 3-C

Driving a 32-Ft. Hydraulic Pressure Tunnel Around the American Niagara. *Engineering News-Record*, Vol. 88, No. 17, April 27, 1922.

The 65 000 Kv-a. Vertical Waterwheel Generator for The Niagara Falls Power Company, by M. C. Olson and E. B. Plunge. *General Electric Review*, Vol. 27, No. 2, February, 1924.

The 65 000 Kv-a. Generator of The Niagara Falls Power Company, by W. J. Foster and A. E. Glass. *Journal*, A. I. E. E., Vol. 43, No. 4, April, 1924.

Three 70 000 H.P. Turbines Installed at Niagara Falls. *Engineering News-Record*, Vol. 94, No. 8, February 19, 1925.

Hydro-Electric Plant Has 452 500 H.P. Installed Nominal Capacity. *Power*, Vol. 61, No. 9, March 3, 1925.

World's Largest Hydraulic Turbines. *Power*, Vol. 61, No. 21, May 26, 1925.

Engineering Consideration in 70 000 H.P. Hydraulic Turbine Design, by H. Birchard Taylor. *Power*, Vol. 61, No. 21, May 26, 1925.

Structural Features of Turbines for Unit No. 21, by W. M. White. *Power*, Vol. 61, No. 21, May 26, 1925.

Three 65 000 Kv-a. Generators at Niagara Falls. *Power*, Vol. 62, No. 3, July 21, 1925.

Edward Dean Adams Station

An Economical Aspect of the Niagara Power Plant, by Arthur Vaughn Abbott. *Electrical Engineering*, Vol. 6, No. 32, August, 1895.

Description of Plant. *Cassier's Magazine*, Niagara Power Number, July, 1905.

Power Development at Niagara, by W. C. Johnson. *Journal*, Assoc. of Eng. Societies, July, 1899.

The Electrical Transmission of Power from Niagara Falls, by Lewis B. Stillwell. *Transactions*, A. I. E. E., Vol. 18, 1901.

Wheel Pit, by O. E. Dunlap. *Engineering News*, Vol. 43, 1, and Vol. 44, 2, 1900.

New Generating Plants, by H. W. Buck. *Engineering News*, Vol. 48, 1, 1902.

Description of Plant. *Engineering News*, Vol. 54, 2, 1905.

The Niagara Falls Power Company Improves Its Niagara Plant, by E. B. Strowger. *Power*, Vol. 66, No. 25, December 20, 1927.

William Birch Rankine Station

Canadian Niagara Power Company New Generating Plants, by W. H. Buck and O. E. Dunlap. *Engineering News*, Vol. 48, 2, 1902.

Construction of Canadian Niagara Power Company's 100 000 Horsepower Hydro-Electric Plant at Niagara Falls, Ontario, by Cecil B. Smith. *Electrical Review,* N. Y., December 2, 1905.

Generating Plant of the Canadian Niagara Power Company. *Canadian Electrical News,* No. 6, June, 1906.

The Hydro-Electric Plant of the Canadian Niagara Power Company. *Canadian Electrical News,* September, 1907.

The Hydro-Electric Development and Transmission Lines of the Canadian Niagara Power Company, by A. H. Van Cleve. *Transactions,* Am. Soc. C. E., Vol. LXII (1909), p. 199.

ONTARIO POWER COMPANY

The Works of the Ontario Power Company of Niagara Falls. *Engineering Record,* Vol. 50, October 8, 15, 22, and 29, 1904.

The Concrete-Steel Headworks of the Ontario Power Company, Niagara Falls. *Engineering Record,* Vol. 51, January 14, 1905.

Stoney Sluices at the Headworks of the Ontario Power Company. *Engineering Record,* Vol. 51, May 20, 1905.

The Intake of the Ontario Power Company. *Engineering Record,* Vol. 52, August 12, 1905.

Hydro-Electric Enterprise in Canada, by Paul N. Nunn. *Canadian Engineer,* March, 1906.

The Development of the Ontario Power Company. *Canadian Electrical News,* No. 6, June, 1906.

Description of Plant. *Mechanical Engineer,* January 7, 1910.

Canadian Commission Buys Ontario Power Company. *Engineering News-Record,* Vol. 78, June 7, 1917.

Annual Report of the Hydro-Electric Power Commission of Ontario, Vol. 1, 1918.

Description of Plant. *Electrical Review,* Vol. 74, November 25, 1919.

Large Generator Bursts at Niagara Plant. *Engineering News-Record,* Vol. 88, No. 18, May 4, 1922.

THE TORONTO POWER COMPANY

Power Development of the Toronto and Niagara Falls Power Company. *Engineering Record,* Vol. 49, February 13, 1904.

Headworks of the Plant of the Toronto and Niagara Falls Power Company, Niagara Falls. *Engineering Record,* Vol. 51, April 8, 1905.

The Niagara Power Plant of the Electrical Development Company of Toronto. *Engineering News,* Vol. 54, November 9 and 30, 1905.

The Hydraulic Features of the Latest Niagara Power Plant. *Engineering News,* Vol. 54, November 30, 1905.

The Electrical Development Company of Ontario. *Canadian Electrical News,* No. 6, June, 1906.

The Power Plant of the Electrical Development Company of Ontario, by F. O. Blackwell. *Electrical Review,* N. Y., July 28, 1906.

Description of Plant. *Canadian Engineer,* Vol. 36, No. 3, January 16, 1919.

Description of Plant. *Electrical News,* Vol. 28, No. 16, October 15, 1919.

QUEENSTON-CHIPPAWA DEVELOPMENT

Description of Plans. *Canadian Engineer,* June 20, 1918.

Canada Rushing Huge Niagara Development as War Conservation Measure. *Engineering News-Record,* Vol. 81, No. 18, October 31, 1918.

Construction Progress on Canadian Niagara Power Project. *Engineering News-Record,* Vol. 83, No. 21, 1919.

Mixing Concrete by Surface Areas on Actual Work, by R. B. Young. *Engineering News-Record,* Vol. 84, No. 1, 1920.

Design of New Canadian Niagara Power Project. *Engineering News-Record,* Vol. 85, No. 16, 1920.

Sliding Form for Lining Chippawa Power Canal. *Engineering News-Record,* Vol. 87, No. 1, 1921.

Cost of Canadian Niagara Hydro-Electric Power. *Engineering News-Record,* Vol. 87, No. 1, 1921.

First Unit for Queenston-Chippawa Water Power Plant Opened. *Engineering News-Record,* Vol. 88, No. 3, 1922.

Placing 410 000 Cu. Yd. of Concrete on Ontario's Niagara Power Development, by A. C. D. Blanchard and R. B. Young. *Engineering News-Record,* Vol. 88, Nos. 14 and 15, 1922.

Queenston-Chippawa Development of the Hydro-Electric Power Commission of Ontario, by F. A. Gaby. *Journal,* A. I. E. E., July, 1922.

Developments of the Hydro-Electric Power Commission of Ontario, by F. A. Gaby. *Transactions,* Am. Soc. C. E., Vol. 90 (1927), p. 883.

Modern Hydraulic Turbines of Large Capacity, by H. G. Acres. *Transactions,* Am. Soc. Mech. Engrs., Vol. 45.

DISCUSSION

THOMAS H. HOGG,[5] M. AM. SOC. C. E.—There are four headings under which the speaker desires to discuss Mr. Gibson's paper, no one of which is closely related to the others. The discussion may appear to be somewhat disconnected, but this is explained by the breadth of the field covered by the author, and the fact that almost every page of the paper opens up avenues of thought, each of which might provide discussion more voluminous than the original paper.

Period of Service and Obsolescence.—The historical notes with regard to the various power developments at Niagara Falls direct one's attention to the life time of the various power plants. Referring to Table 1 it will be seen that the Schoellkopf Plant was commenced in 1852 and completed in 1924. The first of these dates refers to the time at which the construction of the canal that supplied water to this plant was begun, and the last to the completion of Station No. 3-C in this plant. If one goes back to the historical notes themselves, it will be seen that Station No. 1 was built in the period from 1901 to 1904, and gave service for 23 years. Station No. 2, built in the period from 1895 to 1904, with a capacity of 33 900 h.p., was in service from 17 to 26 years, according to the particular unit being considered. These two stations gave place to larger development and have been abandoned and dismantled.

Three other plants, namely, Niagara Falls Power Stations Nos. 1 and 2, and, to a certain extent, the Toronto Power Plant, are in the reserve list; that is, while they are maintained in such a state that they may give service at any time, they are not on full operating schedule. The Toronto Plant does give daily service, but is operated more or less as a peak-load plant, and is shut down at night and frequently on Sundays and holidays. These three plants have been in existence from 20 to 24 years and have given way, to a certain extent, to plants that are more economical in the use of water. There is a third group, comprising Station No. 3-A of the Schoellkopf Plant and the Ontario Power Plant. These plants have been operated for 23 years, or more, and appear to have a great many more years of service before them. The remaining developments have given 6 to 10 years of service, and are not of interest in considering the useful life of a plant, as they are perfectly modern and no appreciable changes have taken place since they were built.

It will be observed that certain plants have gone out of service at about the end of twenty-five years, and certain others have gone into reserve in about the same time. From this one might be led to assume that twenty-five years is about the useful life of these developments. It should be noted, however, that the pioneer plants, which have been dismantled, have only disappeared in part, as the canal used to supply them with water seventy-five years ago is still in service—enlarged, it is true, for greater service—and the advances in demand for power made the space occupied by these pioneers of very great value for enlarged development.

---

[5] Chf. Hydr. Engr., Hydro-Electric Power Comm. of Ontario, Toronto, Ont., Canada.

An examination of the Ontario Power Conduit No. 1 in 1922 by an independent Commission, led to a report that the life time of such a conduit, in contact with Niagara water, would probably be at least 40 or 50 years. The tail-race tunnels of the Toronto Plant and the Edward Dean Adams Plant are apparently in as good condition to-day (1928) as when first put into service, so that one may conclude from experience with the older plants that the water channels have a life time far in excess of the period during which any plant at Niagara Falls has yet operated. It seems reasonable to conclude, therefore, that with proper maintenance, 30 to 50 years of service may be expected from these power developments. For the modern plants, 50 years seems reasonable, because the entire development in each case is so highly efficient that obsolescence can scarcely be effective in causing the abandonment of the plant.

Water Levels on Lake Erie.—The speaker would like to submit a record of water levels on Lake Erie during the storm of December 8, 1927. The author refers to the variation in level on the lake, due to certain winds of high velocity and changes in atmospheric pressure. On the date named, a very strong west wind caused a considerable change in the water level at Buffalo, N. Y., the lake surface rising from 571.1 ft. at 3:00 A. M., to 579.8 ft. at 9:00 A. M. Fig. 13 shows records of maximum and minimum water levels at various points along Lake Erie and on Niagara River during the time of this storm.

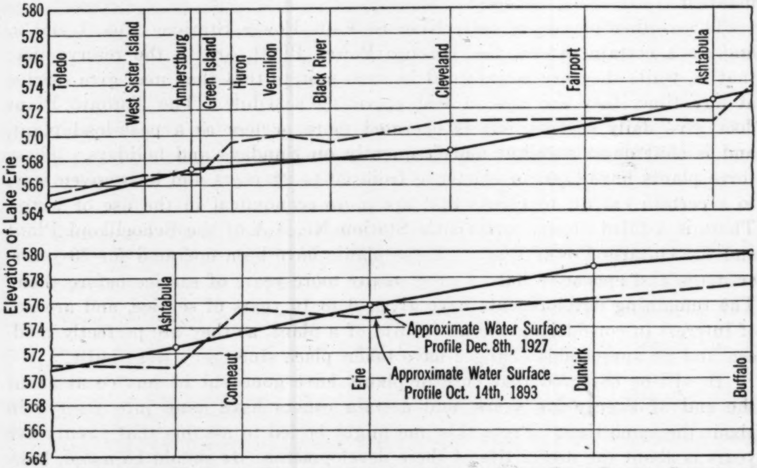


FIG. 13.—PROFILES SHOWING EXTREME OSCILLATIONS OF LAKE ERIE DURING STORMS OF OCTOBER 14, 1893, AND DECEMBER 8, 1927.

These curves indicate a lake surface elevation of 564.5 at Toledo, Ohio, at the west end of the lake, and, at the same time, an elevation of 579.8 at Buffalo, at the east end of the lake. Intermediate readings are consistent with these and show a fairly uniform variation in lake surface elevations from end to end. Fig. 14 gives records of water levels at the various points on

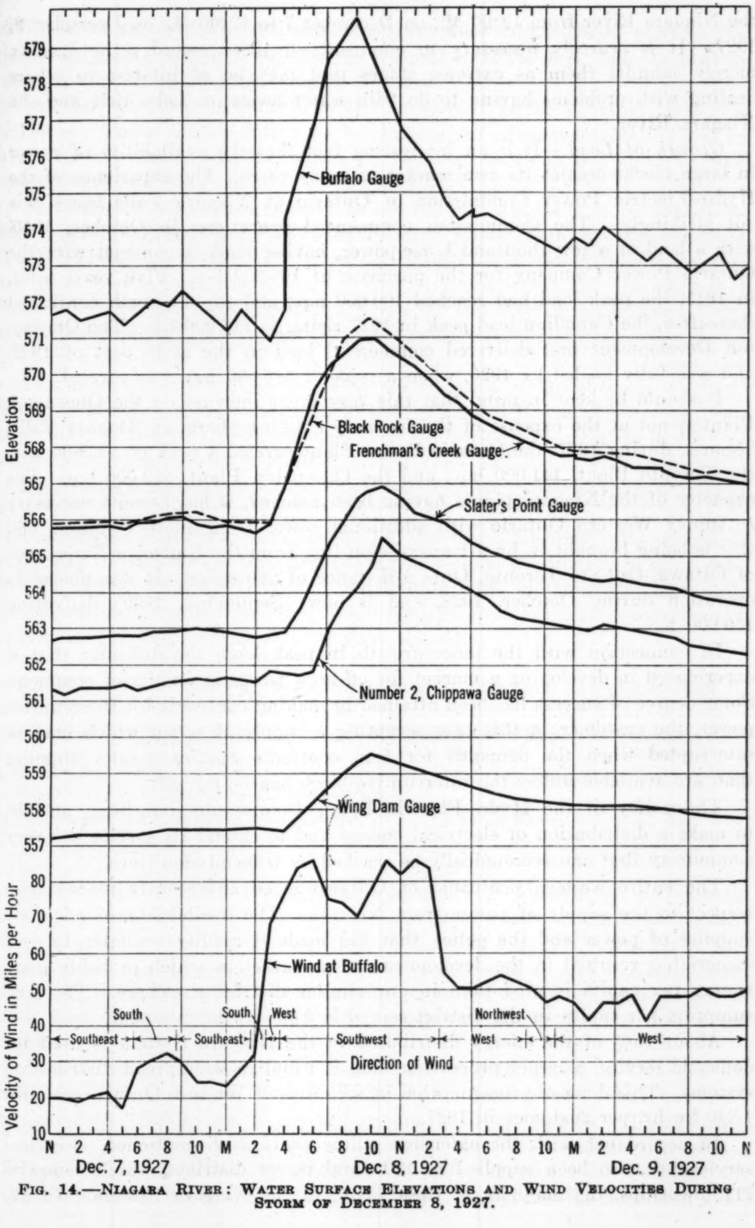


FIG. 14.—NIAGARA RIVER: WATER SURFACE ELEVATIONS AND WIND VELOCITIES DURING STORM OF DECEMBER 8, 1927.

the Niagara River from 12:00 M., on December 7 to 12:00 M., on December 9, 1927. It is scarcely necessary to comment on these records; the speaker merely submits them as extreme stages that may be of interest to others dealing with problems having to do with water levels on Lake Erie and the Niagara River.

Growth of Load.—It is an interesting fact that the availability of power in large blocks creates its own market in many cases. The experience of the Hydro-Electric Power Commission of Ontario at Niagara Falls bears this out strikingly. The Commission commenced operations in October, 1910, with a load of a few thousand horse-power, having made a contract with the Ontario Power Company for the purchase of 100 000 h.p. Five years later, in 1915, the peak load had reached 100 000 h.p., and rapid growth continued thereafter, the Canadian load peak in 1927 rising to 713 000 h.p. The Queenston Development first delivered commercial load in the early part of 1922, and was fully loaded by 1926, when a peak of 525 000 h.p. was carried.

It should be kept in mind that this very great increase on the Queenston Plant is not at the expense of the formerly existing plants at Niagara Falls, because, during 1927, the Ontario Power Plant carried a peak of 182 000 h.p., the Toronto Plant, 141 000 h.p., and the Queenston Plant, 529 000 h.p. The capacity of the Niagara plants having been reached, it has become necessary to supply Western Ontario with additional power from another source, and this is being brought in by a transmission line from the Gatineau River, east of Ottawa, Ont., to Toronto, Ont., a distance of 260 miles. It was placed in operation during October, 1928, and is now (September, 1929) delivering 150 000 h.p.

In connection with the large growth in peak load, the difficulty that is experienced in developing a market for off-peak power is worthy of comment. Some degree of success has been attained in making contracts for interruptive power, the customer in this case accepting a supply of power which may be interrupted when the demands for firm contracts require greater supplies than are available unless the interruptive block can be cut off.

The policy of the Hydro-Electric Power Commission has been, and is, to make a distribution of electrical energy and to extend its service to every community that can economically be reached by transmission lines.

The entire western peninsula of Ontario is advantageously placed with respect to the supply of power from Niagara. The availability of adequate supplies of power and the policy that has made it readily available to consumers has resulted in the development of a market in which probably more energy per capita is used than in any similar district elsewhere. The consumption per capita in the district served is 2 060 kw-hr. per year.

About 15% of the energy distributed in the Niagara System is used for domestic service, a larger proportion than is usual in widespread distribution systems. This domestic consumption in the cities of Western Ontario equalled 1 340 kw-hr. per customer in 1927.

In accordance with the extension policy previously mentioned, electrical service has also been supplied to 120 rural power districts, which comprise 211 townships, the majority of these being in the Niagara System. In all, 9 757 farms and 15 526 other rural customers were supplied with power. The combined peak loads of these systems is equal to 15 000 h.p.; but the benefits derived are out of all proportion to the load.

All fuel supplies for Ontario must be imported, as the Province lacks coal deposits. Under these conditions, decidedly great advantages are derived by the Province from Niagara power in industry, agriculture, and domestic convenience and comforts.

In Table 4 the author shows the capital per kilowatt to be higher in that part of Ontario supplied with Niagara power than in Western New York. There are several differences in conditions of development and distribution that account for this. The largest single source of Niagara power for Western Ontario is at Queenston, where the full head of the Niagara River is developed. This is a type that is more expensive per horse-power than developments around Niagara Falls itself, because of the long channel necessary to convey water from the Chippawa Pool to the plant at Queenston. Furthermore, on account of an imminent shortage of power, it was necessary to proceed with the work on this development at the period of high costs in 1918 to 1921.

In Western New York a larger proportion of the power developed is used at or near the point of production than is the case in Ontario. Thus, there are additional costs for transmission on the Canadian side. Furthermore, the policy in Ontario has been to provide ample supplies of power to small consumers and to rural districts, and distribution costs are thus also increased. A comparison that illustrates this is shown in Table 5, which gives data for the largest corporation serving each of these districts.

TABLE 5.—COMPARISON OF POWER OUTPUT IN ONTARIO AND NEW YORK.

Power Company.	Total kilowatt-hours sold in 1927.	Customers served.
Hydro-Electric Power Commission of Ontario-Niagara System	2 644 528 360	394 567
Buffalo, Niagara and Eastern Power Corporation	4 237 842 392	259 087

Table 5 does not include all power sold either in Ontario or in Western New York, but the figures cover, by far the greater part of the field in each case. The transmission system in Ontario, carrying a larger proportion of the energy sold to a distance from the point of generation, and a distribution system to reach the much larger number of customers served, account for a large part of the difference in cost per kilowatt shown by the author in Table 4.

Recommended Diversions.—The author states, with respect to diversion of water, that "the maximum diversions officially recommended to date, however, are only * * * 40 000 cu. ft. per sec. around the Lower Rapids." This recommendation is contained in a Joint Report of the International Waterways Commission, dated March 3, 1908, which was prompted by a proposal to grant rights to divert that much from the Lower Rapids on the American side of the river. At that time little intensive study had been made of the lower river, and any one estimating the reasonable diversion from that part had very little information to guide him. It was then considered that no more than the quantity named in the report might be diverted without impairing, to a dangerous degree, the ice-carrying capacity of the river. There have been three serious ice jams in the lower river since 1908, and in every case they have commenced at the mouth of the river and then worked up stream. The rapids section of the river remained open or cleared itself of ice quickly, even at periods when very large quantities were running, indicating that this is not the critical part of the channel.

If water is diverted from the Lower Rapids for development of power, it will be returned immediately below the rapids, and so would not adversely affect that part where ice jams have formed; the whole natural flow would still continue in the long critical reach from the rapids to Lake Ontario. Recent and more complete studies than could be made by the International Waterways Commission when the 1908 report was presented, expressing the opinion quoted therefrom by the author, indicate that a much larger diversion than 40 000 cu. ft. per sec. is reasonable. It has always been the contention of Canadians that the diversion of 36 000 cu. ft. per sec., allowed in Canada under the Boundary Waters Treaty, is not limited as to the point of return to the river, the treaty being silent in that regard. The speaker, therefore, cannot agree with the limit suggested by the author.

To complete the data supplied by the author the following unit efficiencies should be added to Figs. 8, 9, and 10, respectively: 67.0%, Toronto Power Plant; 87.5%, Ontario Power Plant; and 91.0%, Queenston Plant.

Frank M. Williams,[6] M. Am. Soc. C. E.—The paper contains considerable interesting historical as well as engineering information. The author has traced, very clearly, successive steps in power development at Niagara Falls which by the end of the Nineteenth Century had progressed to a stage that marked it as one of the outstanding features of engineering achievement in that century.

By a system of diagrams which accompanies the paper, he has traced the progressive stages of development in the various power plants that have been constructed at the Falls. Some of these plants are shown to have been reconstructed or even entirely replaced by more effective developments.

He also has outlined the areas of the United States and Canada to which electric energy generated at the Falls has been transmitted and the benefits which have been obtained, due to the reasonable price of the power thus made available. These benefits are demonstrated by the great manufacturing and industrial developments which have been built up in this territory.

It was not until the advantages to be obtained by the use of this cheap power led to a demand for a greater amount of it that alarm began to be felt as to the injury which was being done, or which might be done, to the scenic beauty of Niagara Falls because of diversion. As is usual in cases of this kind, the remedies suggested were considerably in excess of the danger. These remedies took the form in the first place of legislative acts, which had their bases in the feeling on the part of the admirers of the scenic beauty of the Falls and the adjacent portions of the Niagara River that permanent injury to that beauty was imminent. The Falls of Niagara are so well known to a majority of the people of the United States and to foreign visitors that perhaps it is natural that some such alarm might be felt. However, it seems apparent as a result of the various studies that have been made and the investigations that have taken place that there is not so much danger to be feared from a further diversion, accompanied by certain proposed compensating or equalizing construction in the bed of the river, as there is in the natural recession of the Horseshoe Falls. This recession is likely to continue until the beauty of these Falls is greatly impaired unless a preventive is found in some of the remedial works suggested.

---

[6] Cons. Engr., Albany, N. Y. Mr. Williams died on February 20, 1930.

The more recent trend of events leads to the hope that something of the kind may be attempted in the near future and that in the process of the undertaking a greater diversion for power purposes may be permitted while, at the same time, permanently preserving the features of scenic beauty.

There are complications in accomplishing this, resulting from the fact that several Governmental parties are concerned, namely, the United States, the Dominion of Canada, the Province of Quebec, and the State of New York. The latter has set up a Water Power and Control Commission which has taken the position that the State of New York is the owner of substantial rights at this location.

The problem as to a definite determination of proper balance between the economies to be generally effected by a larger diversion and the element of scenic beauty to be preserved is one of some difficulty, and because of what might be termed a four-party governmental interest, anything that may be accomplished in this respect in the way of greater diversion takes time. This accounts largely for the fact that up to the present, limitations as to the quantity of water to be diverted for power purposes have not been enlarged.

The author suggests that the first step should involve an additional diversion of 20 000 cu. ft. per sec. and that whatever part of it may be developed on the American side should be on a head of 312 ft. He presents data to indicate that the amount of additional diversion at least could safely be made without affecting the permanent beauty of the Falls or of the Gorge. It might even be beneficial in withdrawing from the Canadian side of the river a part of the flow which is causing the recession at the apex of the Horseshoe Falls. This seems a reasonable and sound suggestion. In the speaker's opinion it can safely be made, within the limitations as to diversion which may be arrived at later, when some agreement has been reached as to the character and extent of the proposed remedial works. This should permit of a diversion much in excess of the present total of 56 000 cu. ft. per sec.

There has been some question as to diversions which might be made around the Lower Rapids. This has been under serious contemplation recently by some of the hydro-electric interests, that own riparian rights in this locality. Applications for licenses have been filed with the various Governmental authorities looking toward such diversion. If the total diversion around the Falls is limited to the present amount, then the diversion around the Lower Rapids might well be made. If, however, the total authorized diversion for power purposes around the Falls should reach more than 100 000 cu. ft. per sec., a considerable part of which would be utilized in an over-all development, it might well be found that a diversion around the rapids and the whirlpool is not practicable.

CHARLES KELLER,[7] M. AM. SOC. C. E.—The speaker intends to confine his remarks principally to some amplification of Mr. Gibson's historical account and to discuss the remaining outstanding problem, namely, the formulation of a wise and statesmanlike permanent international policy as to further diversion and development of water power.

The author's account of the manner in which the United States gained control of diversions of water for power purposes on the east side of the Niagara River at and above Niagara, N. Y., does not, perhaps, lay sufficient stress on some details that should be understood by all who are seeking to participate intelligently in the correct decision of questions of public policy.

For example, the National authority over the diversion of water for power purposes at Niagara Falls is, to-day, accepted as a matter of course by most people who have any occasion whatever to think about it. Those who wonder whether, after all, there is any legal justification for the exercise of such authority are few in number, possibly because, among those who are at all interested, it is believed that the Federal Water Power Act has definitely settled the legal status in favor of National control.

Since 1917 there has been a great invasion of the domain of authority formerly supposed to be reserved to the States, but seemingly this has been in keeping with existing public opinion. At any rate, there has been little active objection on the part of the general public.

The case was different before 1910 when the present tendency to broad construction and, sometimes, complete disregard of traditional limitations of constitutional authority, did not exist. Accordingly, when the Burton Act of June 29, 1906, was passed, there was considerable doubt, in Congress and elsewhere, as to its constitutionality. If it were to be considered constitutional, it must derive its authority from the oft-invoked right "to regulate commerce with foreign nations and among the several states and with the Indian tribes".[8]

Notwithstanding this doubt as to jurisdiction, the two power companies submitted to the restrictions of the Act and co-operated, with liberal good will, in the investigations inaugurated by the United States Lake Survey for the purpose of arriving at a conclusion as to the amount of the diversion which, to quote the language of the Act, "* * * shall not injure or interfere with the navigable capacity of the river, its integrity or proper volume as a boundary stream, or the scenic grandeur of Niagara Falls."

At that time, there was no evidence to support an assertion that the power diversions had any effect on "the navigable capacity of the river" and yet this was the only consideration mentioned in the Act that would come within the constitutional grant of power and justify the restriction of power diversions to 15 000 cu. ft. per sec., imposed by the War Department under its terms.

---

[7] Brig.-Gen., U. S. A. (*Retired*); with Byllesby Eng. & Management Corporation, Chicago, Ill.

[8] Section VIII, Paragraph 3, of the Constitution of the United States.

The New York companies, however, acquiesced in accepting the regulatory control of the United States. While, later, the investigation disclosed that three out of the five major power diversions in the United States and Canada did affect the level of Lake Erie and the depths in the Niagara River between Lake Erie and the points of diversion, the effect discernible at that time was small, the proof of its existence perhaps arguable, and the actual material result very slight.

Then, as now, the power companies were good citizens, observant of the decencies that should characterize the relations between citizens and their Government. Indeed, they went further and, in a matter certainly far removed from the National jurisdiction, showed a public spirit as enlightened as it was gratifying. This refers to their voluntary co-operation with the Commission created by the Secretary of War in 1906, for the purpose of remedying unpleasant scenic conditions on the New York side. Certainly, such a matter concerned, not the National or the State, but merely the local, authorities; and yet the two power companies not only accepted advice but originated suggestions for improvement, and, without protest, paid the expense of these scenic betterments. They certainly were pioneers in what is now referred to as "public relations".

Later, came the treaty proclaimed May 13, 1910, certain provisions of which have been mentioned by Mr. Gibson. Article V of this treaty authorized diversions from above the Falls of not exceeding 20 000 cu. ft. per sec. in New York State and 36 000 cu. ft. per sec. within the Province of Ontario, Canada. At that time the restrictions of the Burton Act on diversions in New York State were in force and the two New York companies were permitted to divert, between them, a total of only 15 100 cu. ft. per sec. Notwithstanding repeated efforts by the power companies to secure authority to increase their diversions to the treaty limit, that is, to divert 4 400 cu. ft. per sec. additional, the War Department held that the will of Congress was shown, not by the treaty but by the terms of the Burton Act. Even after that Act expired in 1913, the War Department continued to enforce its limitations rather than those of the treaty. Meanwhile, the three Canadian companies were approaching closer and closer to the treaty limit of 36 000 cu. ft. per sec. laid down for Canada. At this later day, in the perspective of the years that have passed, the speaker is unable to see the wisdom of this unilateral policy of the War Department.

The demands for increased production in Canada created by the World War caused considerable irregularity in, and curtailment of, the supply of power transmitted from the Canadian side into Western New York. At the same time, war orders for the Allies had increased the power load on the New York side to such an extent that the Buffalo General Electric Company, theretofore operating exclusively on Niagara Falls power, decided to build a steam plant. Pending the completion of this plant, this company sought, and obtained from the War Department, permission to increase temporarily the diversion made at the Edward Dean Adams Station. An increase of 1 500 cu. ft. per sec., the equivalent of perhaps 10 000 to 12 000 kw. of additional power for the Buffalo market, was allowed with the understanding that this permission would cease with the completion of the first unit of the steam plant.

Then the United States entered the war, and after that followed the greatest industrial activity the country has ever known. Then, too, came reason to regret the failure of the United States to authorize the fullest possible use of Niagara water for power purposes permitted under the treaty. The supply of water power at Niagara Falls, then about 275 000 h.p., if used exclusively for the manufacture of electro-chemical products essential for war purposes, would probably have been insufficient for war needs, but much of this power was required for the commercial and domestic loads at Buffalo, N. Y., and at points between Buffalo and Niagara Falls. Obviously, it was necessary to continue to supply the normal electric requirements for public and domestic lighting, for street railways, and for countless other purposes incident to the transaction of the daily business of the population, and, until other possibilities were exhausted, good policy demanded the continuance of the temporary permission for increased diversion. In the fall of 1917, the need of energy for war purposes in and near Niagara Falls became so serious that the Secretary of War was forced to intervene and shortly thereafter steps were taken to requisition and to allocate the available power supply in accordance with an orderly system of priority.

The two power companies at Niagara Falls submitted gracefully to this regulation of and interference with their business and did their utmost to promote the war program of the Government. Later, when the agents of the Secretary of War sought means to increase the power supply of this district, an attempt was made to secure legislation authorizing the diversion of 4 400 cu. ft. per sec., the unused portion of the treaty water. Several months were wasted in this effort and, when it became evident that legislation could not be had and that the supply of certain strategic war materials, obtainable only, or chiefly, at Niagara, would probably soon fall short, the War Department finally adopted the only available alternative and, reversing the policy of eight years' standing, decided to issue a revocable license for the utilization of the 4 400 cu. ft. per sec. remaining from the treaty allotment. At about the same time, negotiations were begun, looking toward the consolidation of the two then existing power companies. Here, again, effective progress was dependent on the active co-operation of the two companies and their willingness to expend many millions of dollars, at a time when money was hard to get, on the faith of a revocable permission, an authorization the permanency of which was at least speculative. It is mere justice to those who controlled the two companies to state that they showed no disposition either to haggle or to hesitate and that, when their help seemed necessary to the execution of part of the comprehensive plan for obtaining a power supply adequate for the war program forecast by those in authority, they were prepared to do their share promptly and without asking the financial assistance of the Government.

In short, it took war and war's imperative needs to bring into effective use for the benefit of the people of Western New York the full diversion of water allowed to be made under the 1910 treaty. This appears to be one of the few real advantages that have followed any war.

There may be some, however, who, basing their doubt on the loss of beauty or landscape values, question whether increased diversion from above the Falls results in a net gain to the entire public, and this leads to a brief consideration of the policy that, in the best interest of the entire public, ought to be observed in the future. Mr. Gibson refers to the proposals made by those who, in one capacity or another, have been in a position to offer counsel. These proposals, while differing among themselves, are consistent in that they assume that the beauty of the Falls is of prime importance and must be protected from deterioration due to natural causes and from diminution due to works of Man, including diversions for power purposes. The various proposals range over a wide extreme, there being, on the one hand, those who deprecate all changes in the Falls on the grounds that Nature is more to be trusted than Man and that all changes, for whatever purpose, are likely to be damaging. The proponents of this belief probably regard themselves as the only real conservationists and defenders of the integrity of the Falls. The other extreme is occupied by those who take the utilitarian view that was summarized by the author in the quotation attributed to Lord Kelvin.

Between these extremes are found the opinions of those who feel that Niagara Falls as a spectacle has in the ages suffered more from the unrestrained processes of Nature than it has ever suffered from the works of Man; that natural forces now visibly at work must be neutralized, or their ill effects counteracted; that it is not possible for the human eye or judgment to appraise the relative beauty and impressiveness of the Falls when their discharge varies over a comparatively wide range, a decline of beauty in one aspect being compensated by an enhancement elsewhere; that, most probably, the great appeal of the Falls is based upon a general impression rather than on appreciation of local details; and that, beyond a volume that satisfies the eye and ear of the average visitor, further discharge is neither desirable nor useful. Those who take this middle attitude feel certain that the correct eventual policy will include the utmost amelioration of the appearance of both Falls and the retardation or prevention of the destruction of the Horseshoe Falls, and, as an incident, will provide for such increase in power diversions as is called for by this enhancement of the beauty of the Falls.

The policy will be based upon an orderly program of remedial work, such as that for spreading the discharge over the Horseshoe Falls, followed by observation of results due to varying the power diversions over the widest permissible range and, in turn, by such further remedial and compensating work as may be indicated to assure permanently satisfactory scenic conditions at Niagara and unimpaired channel conditions at Buffalo and on Lake Erie when power diversions have been increased. Those who favor this policy of combining remedial work and improvement of the Falls with the utmost increase in diversion consistent with really significant scenic values are better and broader conservationists than the first-named class because they give thought to the conservation of human happiness and welfare as well as of scenic grandeur.

In this middle school of thought, differences exist as to the ultimate limit of permissible diversion from above the Falls, but all agree that it should be the maximum that can be spared without doing appreciable harm to scenic values. In other words, they feel that power values are almost as important as scenic considerations and that the prosperity and contentment of 5 000 000 permanent residents presents to the unbiased judgment a thoroughly pleasing picture, one that should be weighed in the scale with the casual impressions of 2 000 000 visitors who annually spend a few hours at the Falls.

The speaker's view is identical with that recommended in the report of the Board of Engineers for Rivers and Harbors of 1920, namely, that the total diversion may ultimately reach 100 000 or 110 000 cu. ft. per sec. This is, of course, merely an opinion, but it is based on some familiarity with scenic conditions and with the changes in them during a period when the power diversions have increased from about 30 000 cu. ft. per sec. to nearly 56 000. This report recommended compensating and remedial measures, with step by step increase in diversion and careful observation of the effects of the remedial works and of the progressive increases in diversion. With these remedial measures, it was thought that the maximum diversion mentioned would leave the Falls more beautiful than they had been within the memory of persons now living. Since this report was made, conditions have changed and the rate of diversion may now be varied over a wide range. The effect of the remedial works can be observed, therefore, under greatly varying discharge conditions, and judgment can be formed as to further operations.

Any further development of power should utilize the maximum available head as far as can be done without detriment to the rapids below the Falls. The consensus of opinion is that the limit is a total diversion of about 40 000 cu. ft. per sec. from these rapids, leaving 24 000 cu. ft. per sec. still to be developed, provided it is decided that the existing diversion from above the Falls may properly be increased to that extent. Incidentally, it should be said that the course of the river below the Falls is such that the conduit for such an over-all development on the New York side will be less than one-third the length of the head-race channel of the Queenston Plant. It seems likely, therefore, that if by some statesmanlike agreement, frontiers could be disregarded, the greatest economy could be attained by developing the power of any future over-all diversion in a single plant on the New York side.

Nothing has been said about the engineering features of the six existing water-power developments. They present an interesting picture of progress from the standpoint of mechanical, electric, and hydraulic engineering, culminating in the large units of the Queenston Plant and of Section 3-C of the Schoellkopf Station.

The water power of Niagara Falls is continuous and constitutes an absolutely dependable source of energy backed by the greatest reservoir system known to Man. It is subject to just as little chance of interruption as is steam-generated energy, and it is considerably less costly when delivered on the switchboard available for send-out. Mr. Gibson is conservative in his statement as to the economic value of this particular variety of water power and the speaker feels that, despite the constant improvement in steam-driven machinery, the day will never come when Niagara energy will be superseded.

The case is otherwise with most of the now undeveloped water-power sites in this country. Their development would either result in stream flow plants with discontinuous output or they would require the provision of adequate storage, which would usually be extremely expensive, or supporting steam capacity would be necessary. No great margin exists for further improvement in efficiency of water-power equipment, with water-wheels now producing 94% of the theoretical energy content of the water. On the other hand, steam equipment still has far to go; the efficiency of equipment from the boilers to turbines is certainly not more than 30 per cent. It seems safe to state that many water-power sites of the character just mentioned will probably never be developed, or, at any rate, they will not be put to use until the cost of fuel is much greater than at present.

This makes it clear that, generally speaking, the so-called conservationists and those who make political capital out of undeveloped water power as being the one natural resource still in the possession of the people and who preach that it is desirable to protect this resource from exploitation by the private capital are wide of the mark. If they are sincere, they should rather support a National policy under which these second-rate sites may be developed at the lowest possible cost and thereby promote the conservation of the coal supply of the nation. Obstructions and difficulties placed in the way of the utilization of such sites are detrimental to the general public interest and have already resulted in the abandonment of projects for developing many of them.

William Kelly,[9] M. Am. Soc. C. E.—The author has treated, rather briefly but comprehensively, the question of the regulation of the Great Lakes System. This has been a controversial question for thirty years or more, and one still hears many divergent opinions. The speaker, as a member of the Joint Board of Engineers on the St. Lawrence Project, was called upon to give the subject considerable study. Any one who will read the principal reports that have been published, will probably agree that most of the divergence of opinion is due to incomplete consideration of a very far-reaching problem. The following statements are quite easy to prove, and are generally accepted:

1.—Fluctuation of lake levels can be reduced, but only to a limited extent.

2.—Reduction of the fluctuation of lake levels will decrease the storage available for regulating outflows, and, therefore, will produce greater fluctuation in the flows of the Niagara and St. Lawrence Rivers.

3.—The low-water flow of the St. Lawrence River cannot be reduced, nor can the high-water flow be increased without serious damages.

4.—Fluctuations in the flows of the Niagara and St. Lawrence Rivers can be reduced only by increasing the available storage in the lakes, which means increasing their fluctuations.

5.—The high-water level of the lakes cannot be raised because of the very extensive flowage damages that would result.

---

[9] Col., Corps of Engrs., U. S. A.; Vice-Pres., Buffalo, Niagara & Eastern Power Corporation, Buffalo, N. Y.

6.—The surface of the Great Lakes constitutes about one-third the total drainage area tributary to them. In their natural condition, therefore, they provide so high a degree of regulation that under the law of diminishing returns no great profit can be looked for from greater regulation.

The Great Lakes System covers a vast area and has important effects on many diverse interests in two countries, several States and provinces, and many cities. It is failure to consider the effect of regulation on all these interests that causes most of the disagreement. A few examples may illustrate some of the difficulties:

(*a*) If the fluctuations of Lakes Michigan, Huron, and Erie are reduced, the fluctuations of the flow of the Niagara River will be increased to the detriment of power and navigation interests on that river; the fluctuations of Lake Ontario will be increased to the detriment of riparian owners and of navigation thereon; and the fluctuations of the St. Lawrence will be increased to the detriment of riparian owners and navigation above Quebec, Que., and of power interests above Montreal, Ont., Canada. Calculations show that the damages far outweigh the benefits.

(*b*) Mr. Gibson shows that an attempt to regulate the flow of the Niagara River so as to keep the minimum flow from falling below 180 000 cu. ft. per sec., which has been proposed as an aid to power development, would result in the production of about 2 000 000 000 kw-hr. per year, which is less power than can be obtained from the natural flow available 50% of the time. At the low figure of 2.8 mills per kw-hr., this loss would capitalize at about $100 000 000 and, at the cost of steam power, which would eventually have to replace it, the capitalized loss would be more than $200 000 000. The Joint Board of Engineers on the St. Lawrence Project found that regulating the flow of the St. Lawrence to increase the minimum flow would have a similar effect upon power developments on the St. Lawrence.

Many other examples of the difficulties involved in regulation can be given, but those who are not convinced by Examples (*a*) and (*b*) probably cannot be convinced unless they make complete studies themselves, carrying them far enough to determine what will result from their proposed regulation all the way down to Quebec on the St. Lawrence River. The Joint Board made very laborious computations to determine the effects throughout the Great Lakes System of all the practicable plans of regulation that had been proposed. As a result, the Board found that:

> "The regulation of Lake Superior has been satisfactory to the two countries for the reason that the fluctuations introduced in discharge from that lake are absorbed in the great reservoir formed by Lakes Huron and Michigan without greatly affecting the levels of the latter or materially affecting the discharge of the Niagara and the St. Lawrence Rivers."

* * * * * *

> "The regulation of Lake Ontario, proposed as a necessary part of the improvement of the St. Lawrence, affects only Lake Ontario. Its regulation will not affect, in any substantial manner, divergent National interests."

* * * * * *

> "The regulation of the lakes, as a whole, is an entirely different matter."

While the Board was unable to find any economically feasible method of regulating the outlets of Lakes Michigan, Huron, and Erie, it did find it "advisable to construct compensating works in the Niagara and St. Clair Rivers to counteract the effect of all diversions and outlet enlargements on the levels of Lakes Michigan, Huron, and Erie."

R. D. JOHNSON,[10] ESQ. (by letter).—The author has touched upon the direct benefit accruing to a large population and the indirect benefit to the whole country through the distribution of Niagara power, which is sold much cheaper than the average in other territories less favorably situated. He has effectively disposed of a misconception that power development at Niagara Falls has affected the level of Lake Erie. He makes clear the difficulty, if not the futility, of attempting to regulate the lake level either to diminish its fluctuations or to improve the flow of the Niagara River, but he does point out the great desirability of modifying the discharge over the Horseshoe Falls and increasing the flow over the American Falls, both as an assurance of the preservation of scenic values and as a means of releasing additional water for power purposes. A very important point in his recommendation is the fact that this work may progress gradually, testing each phase of the situation by measurements and observation, so that there is not the slightest danger of having to depend solely upon pre-determined engineering computations for the ultimate result.

Mr. Gibson's values of revenue received from the sale of Niagara power brings to mind immediately a popular misconception as to the intrinsic value of undeveloped water power in the minds of many, although it is believed their number is decreasing. A notion seems to have grown up that water power costs very little because Nature so easily furnishes the water as compared, for example, to the arduous mining and shipping of coal. It is this naive idea, perhaps, that has been largely responsible for a great deal of Socialistic propaganda that the natural water power belongs to the people and that private interests should at least be heavily taxed by the Commonwealth for the privilege of developing and distributing such power. A water-power site, in general, only becomes valuable and able to compete with other kinds of power when: (1) The topography favors development; (2) the flow is dependable; (3) the very best of engineering talent is brought into play; and (4) the location is not too far from the power market.

Niagara power has the benefit, to a marked degree, of these four requisites, but it should not be forgotten that its great success to-day is due to the highest quality of engineering skill.

While it is true that, when a favorable water-power site has been skilfully developed in the vicinity of a proper market, it may deliver cheap power, yet it is quite as true that even a favorable site may cost so much to develop, because of ineffective design and blunders, to say nothing of the great risk always involved in corralling and handling large volumes of water, that the perpetual interest charge will run the cost above that of other kinds of power. It is clear to those well versed in this subject that private enterprise is just as proper and effective for the public good in water-power development as in any other branch of public utilities. Even what might be regarded as a simple extension of the present Niagara power development could easily be almost ruined from an economic standpoint if not undertaken by those who have spent their lives at this business and in that locality.

---

[10] Hydr. Engr., New York, N. Y.

To sum up, then, it should be understood that the great economic benefit to the whole population of the further development of the remaining available water-power resources is only possible to the fullest extent when carried out with the strictest competitive economy such as is well known to result from private enterprise. That the benefit of such economy is necessarily passed on, either directly or indirectly, to all the people is an important point of Mr. Gibson's paper.

GLEN E. EDGERTON,[11] M. AM. SOC. C. E. (by letter).—The author has pointed out clearly the prominent place that the Niagara Falls power plants have taken in the development of the art of hydro-electric power generation. The utilization of this natural resource is not yet complete, and its problems will continue to occupy a conspicuous position in the future as they have in the past. Great credit is due the individuals who have contributed to the solution of these problems. Although such credit may not have been fully accorded, it is significant that many of the most prominent engineers with whom the writer has come in contact, in connection with water power development throughout the United States, have been associated at some time in their careers with the Niagara Falls development.

The pre-eminence of Niagara Falls power in the water-power industry is paralleled by its prominence in the affairs of the Federal Power Commission. The first license issued under the Federal Water Power Act of June 10, 1920, covered the existing plants on the American side. The application for a license was submitted originally on July 1, 1920, before the Commission was fully organized for the transaction of business. Nine other applications for power development involving the Niagara River were submitted to the Commission, which gave them final consideration at a meeting of February 28, 1921, and authorized a license to be issued to The Niagara Falls Power Company for the then existing plants and the enlargment of the Schoellkopf Station to its present capacity.

The deliberations of the Commission in these cases involved some important features of the new water policy. Among them was the practical application of Section 10 (*a*), of the Federal Water Power Act, which reads as follows:

"Sec. 10. That all licenses issued under this act shall be on the following conditions:

"(*a*) That the project adopted, including the maps, plans, and specifications, shall be such as in the judgment of the commission will be best adapted to a comprehensive scheme of improvement and utilization for the purposes of navigation, of water-power development, and of other beneficial public uses; and if necessary in order to secure such scheme the commission shall have authority to require the modification of any project and of the plans and specifications of the project works before approval."

---

[11] Major, Corps of Engrs., U. S. A.; Dist. Engr., U. S. Engr. Office, Rock Island, Ill.; formerly Chf. Engr., Federal Power Comm., Washington, D. C.

Before authorizing any project, it is incumbent on the Commission to determine whether it is "best adapted to a comprehensive scheme of improvement and utilization." While The Niagara Falls Power Company did present alternative schemes, including several types of over-all development, the application dealt primarily with the 215 ft. of head obtainable by diversion around the Falls only. Manifestly, this project did not of itself constitute a comprehensive development for the Niagara River to the extent of permissible diversions of water, because it left undeveloped the fall in the Lower Rapids, amounting to nearly 100 ft. It had already been established that a large quantity of water could be diverted around these rapids without injury.

The probable feasibility of over-all diversion was indicated not only by the alternative proposals of The Niagara Falls Power Company, but by two other applicants for authority to develop the total fall. The Commission, then, had at its disposal 20 000 cu. ft. per sec. of water which it could authorize for use in: (1) A development through 215 ft. of fall around the Falls, leaving the remaining fall for separate development; (2) a single development through about 310 ft.; and (3) a development using part of the water around the Falls only, with an over-all development for the remainder.

Investigation showed that the cost of development around the Falls alone gave the cheapest power. Capital costs per horse power of over-all development in a single stage would exceed those of a Falls development by about 40 per cent. Similar costs of the second stage, in a two-stage development, and the cost of the entire development in the two stages, would exceed the same figure by about 250% and 125%, respectively. Consequently, authorization of the Falls development would produce much the cheapest power immediately, but it would commit the Commission to an eventual scheme of development much more expensive than the best development, unless the latter should be postponed until the power plants at the Falls could be scrapped.

It appeared that the partial development proposed was capable of meeting the power requirements of the tributary region for several years; that the more expensive development, although economically feasible, would necessarily involve higher charges for power to the disadvantage of industries which, in some cases, owed their existence to the possibility of procuring power at minimum rates; that additional power was needed urgently and could be supplied most promptly by a partial development; that probably additional diversions could soon be authorized in connection with remedial works at the Falls so as to permit a second step in development which might embrace the entire 310 ft.; that such a step would serve the urgent needs of the power market for an extended period; and that even in the ultimate development it will probably be possible to permit greater diversion around the Falls than around the Lower Rapids on account of the requirements of the ice-carrying capacity in the latter. The relative degrees to which these and similar factors governed the final decision of the Commission can not be stated precisely, but the conclusion was reached that the Falls development proposed in the Schoellkopf Station was best adapted to a comprehensive scheme of improvement.

A license was issued on March 2, 1921, to permit the use of 19 500 cu. ft. per sec. of water, the remainder being reserved to satisfy equitable rights that seemed to be vested in certain power users along Eighteen-Mile Creek below Lockport, N. Y., by reason of long use. Of this water, 225 cu. ft. per sec., which was subsequently determined to be in excess of the quantity required to satisfy those rights, was allotted to The Niagara Falls Power Company on December 13, 1926; and since rights themselves have now been acquired in the interest of that Company, it is expected that the entire quantity authorized by the Boundary Waters Treaty will shortly be available for the Schoellkopf Station.

This case illustrates the necessity for broad and reasonable interpretation of one section (and the same necessity applies to other sections) of the Federal Water Power Act. Little criticism of the principal conclusion is possible, but it might have been held that complete physical development in the most economical way should be required immediately. The results of such a finding would have violated the economics of the situation as it existed in 1921. The reservation of a quantity of water for relatively wasteful use at Lockport and along Eighteen-Mile Creek can not be so easily established in accordance with the principles of the law.

A limitation of the maximum rate of diversion to 120% of the average rate was fixed by the Commission in its license for the American plants. The pertinent words of the treaty are, "not exceeding in the aggregate a daily diversion at the rate of twenty thousand cubic feet of water per second." If it was intended to fix the maximum rate at 20 000 cu. ft. per sec., there seems to be no significance to the word, "daily." If it was desired to fix only the quantity to be diverted in each day, it might readily have been stated definitely in acre-feet or other convenient units. There is room for disagreement as to the exact force of the phrase, but the Commission, concluding that a higher rate than 20 000 cu. ft. per sec. was permissible, also believed that the public interest required a limit which would not permit diversion at a very high rate for a few hours each day. No such specific limitation is known to govern the plants on the Canadian side, which, the writer believes, actually operate with a peak somewhat higher than 20% above the average rate of diversion.

The Federal Power Commission undertakes to preserve natural scenic grandeur and beauty wherever they are of importance in connection with any prospective license. This responsibility is considered to be a part of the obligation, laid down in the section of the Federal Water Power Act previously quoted, to insure provision for other beneficial public uses in addition to power development and navigation. Such questions have been prominent in many water-power projects beside those at Niagara Falls. Developments on Lake Chelan, in Washington; Wolf River, in Wisconsin; Cumberland River, at Cumberland Falls, in Kentucky; Arkansas River, at Royal Gorge, in Colorado; and the Potomac River, at Great Falls, in Maryland and Virginia, are conspicuous cases in which strenuous objections to the issuance of a license have been made by organizations and individuals, concerned lest the Commission might fail to recognize or to perform effectively its duty in such matters.

In the Niagara Falls case it was not considered that further studies, in addition to those which had resulted in the treaty provisions, were necessary to establish the desirability of permitting the diversions approved therein, but an article was included in the license to require the power stations to be constructed and maintained so as to blend harmoniously with the scenic features near them.

F. P. Williams,[12] M. Am. Soc. C. E. (by letter).—The outline of the power plants given in the paper not only shows the progress in the industry at Niagara Falls, but illustrates the tendency in other places to consolidate the generation of power at one location in one large plant. If space had permitted, Mr. Gibson would probably have described the difficulties, especially with bearings, in the early days which prevented development of the total head. There are many who will recall seeing the tail-water of some of the plants discharging half way down the cliff.

Limitation of Diversion of Water Under the Existing Treaty.—In mentioning the Chicago diversion as being one of the factors that led to the unequal diversion allotted under the treaty, Mr. Gibson doubtless gives weight to the discussions leading up to the ratification of the treaty. The treaty itself does not mention the Chicago situation. From evidence presented in the Courts, it appears that the large diversion made by the Sanitary District is illegal. Construction of works to take care of the sewage of Chicago and to relieve this condition is in progress in order to reduce the quantity to the necessities of navigation. The flow required for navigation purposes is being determined by the Supreme Court of the United States. In view of this it would appear that consideration of a greater diversion of water at Chicago than that required for navigation was without legal foundation.

Economic Benefits.—Mr. Gibson's view that steam improvements have not advanced to a point that will enable all water-power plants to be discarded is well reinforced by the construction of such plants that is taking place at the present time (1929) in Canada, in some parts of this country, and abroad, especially in Switzerland. It is evident that there are some factors besides the comparison with steam-generating cost which determine the feasibility of hydro-electric projects. Stability of water power, which enables its cost to be computed accurately for a term of years, and freedom from fear of interruption which confronts steam-produced power on account of possible interference with fuel supply, are among such factors. In some cases these influences are of great importance.

Further development of power at Niagara is feasible, providing works are constructed in the river to distribute the flow; the development of water power on the St. Lawrence River is economic; additional regulation by storage reservoirs will result in more hydro-electric projects being developed on the interior streams.

The statement that the company had not collected the difference between the cost of steam and hydro-electric power from the consumer, but a considerably less sum, calls attention to the industries that have been built up at Niagara Falls which require low cost power in order to exist. Until recently Niagara Falls was the center of the largest electro-chemical metallurgical industries on this Continent.

---

[12] Secy. Engr., New York Water Power Comm., Albany, N. Y.

Ultimate Diversion.—Mr. Gibson has recognized the universal solicitude for the preservation of the scenic grandeur of the Falls. Credit for their preservation is due to the establishment of parks on both sides of the river by Government agencies; to the treaty for limiting the amount of diversion; and to the architects and engineers for the design of works so as to subordinate them and harmonize them with the surroundings as far as possible.

The author describes the plan for increasing the diversion sufficiently for an economic development rather than attempting to determine the ultimate permissible diversion. The preservation of the grandeur of the Falls appears assured as long as there is as conservative a plan for increased diversion and as long as there is as frank a presentation of the various influences.

LEWIS B. STILLWELL,[13] M. AM. SOC. C. E. (by letter).—In this comprehensive paper the author refers to the question: "What led to the adoption of 25 cycles as the standard frequency of alternating current generated at Niagara Falls?" In answer, he quotes a paragraph from the "Electrical Engineering Papers" of the late B. G. Lamme. Mr. Lamme's statement is correct as far as it goes, but further explanation of the origin of the two standard frequencies now in wide use in America, namely, 60 cycles per sec. and 25 cycles per sec., may be of interest.

The frequency originally adopted by the Westinghouse Electric Company in 1885, when that Company actively undertook the development of alternating current for commercial purposes, was 133⅓ cycles per sec. This was the frequency used by Gaulard and Gibbs in demonstrating their system in London, and the original Gaulard transformers imported by Westinghouse were designed for it. A few years later the advantages of direct connection of engine and alternator were clearly recognized, and it became necessary to select one or two lower frequencies adapted to the lower alternator speeds which direct connection implied. In selecting a lower frequency, it was necessary to consider its effect upon transformers, motors, incandescent lamps, and arc lights as well as upon the cost and performance of the alternators. Tesla's American patents covering the polyphase motor had been acquired in 1889 and serious difficulties had been encountered in trying to adapt this motor to 133⅓ cycles.

Experiments demonstrated that American arc-light carbons of that date would not give satisfactory results on frequencies lower than about 60 cycles and after a considerable period of study and experimental investigation, it was decided in 1892 to adopt 60 cycles per sec. as the Company's standard frequency for installations in which incandescent lighting, arc lighting, and motors for industrial purposes predominated. Where a large proportion of the power transmitted by alternating current must be converted to direct current for street railway or storage battery uses, it was decided to use 30 cycles.

---

[13] Cons. Engr., New York, N. Y.

At the time these frequencies were adopted and, in fact, for many years afterward, converters that would operate successfully on 60 cycles had not been produced. This was the controlling reason which led to the adoption of 30 cycles as an alternative frequency notwithstanding the fact that the engineers of the Company clearly recognized the advantages of a single standard.

In 1893, when formal proposals for the first three 5 000-h.p. alternators and auxiliary equipment were invited by the Cataract Construction Company, the companies submitting tenders were faced by the fact that the hydraulic turbines had been ordered and would operate at 250 rev. per min. As it was found impracticable to design two-phase alternators which, at this speed, would deliver current at 30 cycles, the first formal tender of the Westinghouse Company was for 16-pole machines which would operate at 33⅓ cycles per sec. Professor George Forbes, one of the consulting advisers of the Cataract Company, advocated 16⅔ cycles.

In the early autumn of 1893 the writer was asked by the then President of the Cataract Company, Edward Dean Adams, F. Am. Soc. C. E., whether there was any frequency other than 133⅓ cycles per sec. upon which the Westinghouse Company would be willing to bid. In response the writer expressed the opinion that 25 cycles was practicable. Mr. Adams asked him to take the matter up with his associates at Pittsburgh, and the result was a new tender based upon 25-cycle equipment. This tender was accepted by the Cataract Construction Company, and, in this way, 25 cycles was adopted as the standard frequency at Niagara.

In connection with the first alternators, it was necessary, of course, to design transformers, meters, volt-meters, watt-meters, etc., for the same frequency, that is, 25 cycles per sec.; thereafter in practically every case in which the Company undertook to supply alternators and auxiliary equipment for a frequency lower than 60 cycles per sec., 25 cycles became the standard.

In this connection, it may be pointed out that the power transmission exhibit of the Westinghouse Company at the World Columbian Exposition in Chicago, Ill., which was designed and constructed during the autumn of 1892 and the following winter, operated at 30 cycles per sec. This exhibit included a 375-kw., two-phase alternator, step-up transformers, a short transmission circuit, step-down transformers, rotary converters supplying direct-current motors and other direct-current apparatus, two-phase induction motors, and arc and incandescent lamps. In every essential, except frequency, it was the system adopted by the Cataract Construction Company. That 25 cycles and not 30 cycles was used at Niagara was due to the fact that the Cataract Construction Company had committed itself to a turbine speed of 250 rev. per min. and to the fact that one of the advisers of the Cataract Construction Company preferred a still lower frequency. In the writer's judgment experience has demonstrated that 30 cycles would have been preferable, especially in view of the fact that at 25 cycles incandescent-lamp service is not always satisfactory. Recognition of this possible difficulty was one of the reasons upon which the engineers of the Westinghouse Company based their preference for 30 cycles.

Norman R. Gibson, M. Am. Soc. C. E. (by letter).—Since the discussions were for the most part additions to, rather than criticisms of, the writer's statements, there seems little need for reply other than to thank those who have so ably contributed and to express appreciation for the information they have submitted.

Mr. Hogg's interesting charts showing the magnitude and rate of daily variation in Lake Erie and Niagara River levels caused by wind, illustrate the difficulties to be solved in the problem of lake regulation. The information he has given in regard to the efficiency of Canadian plants, the growth of the Ontario power load, and the period of service and obsolescence of the plants, is an important addition to the facts concerning Niagara power.

The discussion by General Keller has amplified the historical account of power development at Niagara Falls and has explained clearly the provisions for the control of diversions from the river for power purposes. The writer is heartily in accord with the suggested orderly program "of combining remedial work and improvement of the Falls with the utmost increase in diversion consistent with really significant scenic values."

The readers of this paper will be particularly grateful to Colonel Kelly for his discussion which submits in brief form the results of a careful study of the problem of lake regulation which was undertaken by the Joint Board of Engineers. The writer is in accord with the conclusion of this Board that it is advisable only to construct compensating works to counteract the effect of all diversions and outlet enlargements on the level of Lake Erie.

Mr. Johnson has given further evidence of the economic value of water power and that these values are necessarily passed on to all consumers.

The discussions by Major Edgerton and Mr. F. P. Williams serve to indicate the viewpoint of those charged with the control of water power development under Federal and State authorities, respectively. Both point the way to early and effective co-operation in the economic development of water powers.

The writer is indebted to Mr. Stillwell for his further explanation of the origin of 25 cycles as the standard frequency of alternating current generated at Niagara Falls. As Mr. Stillwell, himself, played an important part in the early development of Niagara power, it is of a special benefit to have his authoritative statement on this question written into the record.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1764

REGULATION OF LEVELS, FLOW, AND NAVIGATION, NIAGARA RIVER:

SUMMARY AND CONCLUSIONS OF VARIOUS STUDIES[1]

By George B. Pillsbury,[2] M. Am. Soc. C. E.

With Discussion by Messrs. Robert E. Horton, Lynn L. Davis, Theron M. Ripley, E. W. Lane, F. G. Ray, and George B. Pillsbury

Synopsis

The factors relating to the regulation of the Niagara River are summarized, and the studies made by various agencies of the regulation of Lake Erie are described. The conclusion is reached that the results of these studies have not indicated the advisability of regulating the flow of the Niagara River.

The purpose of this paper is to discuss the regulation of the Niagara River, with its widespread ramifications, and to relate the efforts that have been made to solve it. Briefly, the problem is this: To find in what manner the natural regimen of the outflow from Lake Erie through the Niagara River can be altered to produce benefits to mankind which will so outweigh the resulting injuries as to warrant the cost of constructing and operating the controlling works.

There is certainly no insuperable engineering difficulty in building a gate structure that will control the outflow of the Niagara. The problem is how to use it if built.

Inflow, Lake Level, and Discharge

The outflow of Lake Erie is now determined by the discharge over the natural rock weir of the rapids at the head of the Niagara River. While this is a submerged weir, the back-water is almost entirely a function of the head, and the discharge varies with the elevation of Lake Erie at Buffalo, N. Y. Within the comparatively narrow limits of range of stage of the lake, the discharge varies substantially as the lake elevation, thus establishing the present balance between the storage in the lake and the outflow. The increasing discharge gradually checks an undue rise in head from an excessive supply of water to the lake, and the decreasing discharge gradually checks an undue fall when the supply is deficient. At the same time the lake absorbs in storage any temporary fluctuation of supply.

---

[1] Presented at the meeting of the Waterways Division, Buffalo, N. Y., July 19, 1928.

[2] Brig.-Gen., U. S. A.; Asst. Chf. of Engrs., U. S. A., Washington, D. C.

The relation between storage and discharge for Lake Erie may be expressed by the statement that a rise of 1 ft. in the lake will create additional storage equivalent to a flow of 110 000 cu. ft. per sec. for 1 month; and will increase the discharge out of the lake by from 20 000 to 23 000 cu. ft. per sec., depending on the stage.

As a consequence of the present automatic regulation, the extreme variation in the monthly mean levels of Lake Erie, in the sixty-eight years for which continuous and authentic records are available, has been 4.13 ft. The highest monthly mean level was reached in June, 1876; the lowest in December, 1925. During the intervening period the lake levels had been lowered by 0.6 ft., on account of the diversions made by the Chicago Sanitary District, the Welland Canal, and otherwise. Therefore, the fluctuation of the monthly mean elevation of the lake, due to varying supply, has been only 3.5 ft. The corresponding variation in the monthly mean discharge of the Niagara River has swung from a maximum of 252 000 cu. ft. per sec., in June, 1876, to a minimum of 166 000 cu. ft. per sec., in December, 1925.

These lake levels and discharges are monthly means. The water elevation at Buffalo is particularly subject to fluctuation, due to wind and barometric pressure. During a westerly gale the water has been known to rise 8 ft. above its monthly mean level; and, again, it has fallen 4 ft. below its monthly mean. During such periods, discharge of the Niagara River as low as perhaps 110 000 cu. ft. per sec., and as high as more than 300 000 cu. ft. per sec., may obtain.

The variations in the monthly mean levels of the lake are due to fluctuations in supply, that is, to the variations in the sum of the water received from the upper lakes and the run-off of the Lake Erie Basin, less the evaporation. The maximum monthly supply of record to Lake Erie was in April, 1913, corresponding to the rainfall which created the disastrous floods in the tributaries of the Ohio in March of that year. It reached an average of 324 000 cu. ft. per sec. for a month and resulted in a rise of lake level of about 1.6 ft. During August, September, October, and November, the evaporation from Lake Erie usually exceeds the combined rainfall on the lake and the run-off from its own drainage basin, but the supply is well maintained by the inflow from Lake Huron. The minimum supply appears to have occurred in January, 1920, due to the retardation of the inflow by ice obstruction in the St. Clair and Detroit Rivers. It amounted to 126 000 cu. ft. per sec., and was accompanied by a fall of about 0.5 ft. in level.

The total supply to Lake Erie is usually greatest in April and least in October. The April supply exceeds that of October on the average by about 80 000 cu. ft. per sec. The resulting seasonal fluctuation in the level of the lake averages about 1.5 ft., and the corresponding average seasonal fluctuation in discharge, about 30 000 cu. ft. per sec.

These are the high points of the variations in the discharge of the Niagara, with its present natural regimen, and of the resulting levels of Lake Erie.

INTERESTS AFFECTED BY REGULATION

Before considering schemes for regulation it is necessary to cast up an account of those who are affected by the discharge of the Niagara River. Since this discharge controls the level of Lake Erie, these include the navigation interests on that lake. Such navigation is a vital part of the general lake traffic. It moves yearly about 45 000 000 tons of iron ore into ports on Lake Erie, or most of that used in the iron industry on the Continent; it moves about 300 000 000 bushels of grain annually; and it moves the coal shipped from Lake Erie to the territory lying north and west of the lakes. Lake commerce is highly organized, and utilizes practically the last inch of depth available. While it is true that the limiting draft, in recent years, has been fixed by the levels of the upper lakes, no system of regulation could be tolerated which proposed to lower either the extreme low, or the average, levels of Lake Erie. On the contrary, studies heretofore made of the regulation of the Niagara have been directed primarily to improving the lake levels.

A second set of citizens of the United States and Canada vitally affected by the levels of Lake Erie is the several million residents of the cities and towns on the shores of the lake, and along the Detroit River. They are interested, primarily, in the high lake levels. The sewer systems of the low-lying districts of the cities are based on the levels of the lake as they exist at present, and would be inadequate if these levels were raised, or even if they were maintained continuously at the highest levels occasionally reached in the past. A large industrial area in Buffalo is subject to overflow at rare intervals by the extreme storm rises of the lake. If the lake levels were substantially raised, the losses would become more frequent and more extensive. It is certain that no substantial increase in the high levels of the lake, either in height or in frequency, would be tolerated.

The third class very directly concerned in the regulation of the Niagara is the vast number of those who come to enjoy the scenic beauty of the Falls, as well as those, on both sides of the border, concerned in the development of power. Their interest in an unimpaired discharge of the Niagara is too obvious to require elaboration. It is probably more important to them that what may be termed the ordinary flow be not reduced, than that occasional extreme low flows be improved.

A fourth group are those on Lake Ontario and on the St. Lawrence. Probably the most important are the navigation interests in the Port of Montreal, Que., Canada. The low-water elevation in that port is 18 ft. or more, above sea level, and depends on the slope of the St. Lawrence below Montreal, which, in turn, is maintained by the volume of discharge. While it is true that the storage in Lake Ontario will "damp out" any temporary fluctuations in the outflow of the Niagara, yet any systematic retardation, continuing over several months, will be felt at Montreal. It is not enough that the minimum flow be kept at a point higher than that to which it has actually fallen in the past; the normal average low-water flow during August, September, October, and November must be maintained. If the normal flow of the Upper St. Lawrence is reduced, an abnormal low discharge of the Ottawa River may result in record low levels in the harbor.

An increased discharge of the Niagara in winter may affect the flood heights during the spring break-up at and below Montreal. Large sums have been expended to guard against the consequences of these floods.

The future development of power, in a comprehensive way, on the St. Lawrence above Montreal will entail the control of the outflow of Lake Ontario, through the turbines of power plants and the sluice-gates of dams. It is clear that this control would be affected, perhaps favorably and perhaps unfavorably, by variations in the inflow into Lake Ontario through the regulation of the Niagara.

In short, many thousands of citizens of the United States and Canada, extending from Detroit, Mich., to Quebec, Que., have based their industry and their mode of existence, to varying degree, on the regimen of the Niagara as now established by Nature. The effect of any change must be considered before a scheme for regulation can be accepted by the two Governments concerned. If the direct benefits prove large and certain, a way will eventually be found to overcome the resistance of those who will be, or fancy they may be, injured to some degree directly or indirectly. If, on the other hand, the direct benefits are small, and realizable by other means at a cost commensurate with the cost of regulation, then it is evident that widespread indirect disadvantages will throw the balance against regulation.

An Error of Viewpoint

A misconception exists as to the extent of the improvement in lake levels that might be accomplished by regulating works. The diversion of the Chicago Sanitary District may be placed at 8 500 cu. ft. per sec., that through the Welland Canal (Niagara River) at 3 000 cu. ft. per sec., and the equivalent effective diversion through the Black Rock Canal in the Niagara at 1 000 cu. ft. per sec., a total of 12 500 cu. ft. per sec. The result of these diversions is that the levels of Lake Erie are steadily 0.6 ft. lower than they would be had the diversions not been made. By constructing fixed works which will so contract the Upper Niagara River as to reduce its discharge capacity by 12 500 cu. ft. per sec. at all stages, the loss of 0.6 ft. can be restored without otherwise controlling the discharge. In other words, the low level of the lake, occurring subsequent to the diversions, would be raised by 0.6 ft. without increasing the high levels, which occurred prior to the diversions.

When, therefore, a system of regulating works is studied, and is found with the supplies of record, to raise the low levels of the lake by, say, 1.6 ft. without raising the high levels of record, it is evident that the 0.6 of the amount should not be credited to the regulation as such, but is the equivalent of a constant retardation which could equally well be secured by fixed works, at far less cost. The benefit attributable in such case to the installation of manually controlled gates is 1.0 ft. and not 1.6 ft.

Official Studies

Apparently, the first serious proposal for the regulation of Lake Erie was made in 1900 by the Board on Deep Waterways, which had been specially appointed to prepare plans and estimates for securing deep-draft navigation from the Great Lakes to New York Harbor. This Board proposed gates at the head of the Niagara River, which would hold Lake Erie at substantially the level of 574.7. This elevation is slightly higher than the maximum monthly mean level since 1860. It is only once in twenty years that the lake reaches within a foot of this elevation under the natural regimen of the Niagara River. It is very clear that so high a level would not now be tolerated.

Lake Erie can be held at a fixed level solely by discharging water from it as fast, and only as fast, as it is received. The discharge capacity of the Niagara, neglecting the retardation due to any gate piers, is 257 600 cu. ft. per sec. when Lake Erie is at Elevation 574.7. The supply for March and April, 1913 (296 000 and 324 000 cu. ft. per sec., respectively), would have raised the lake by nearly a foot with all regulating gates wide open. It is not possible to discharge water out of Lake Erie as fast as it may be received, unless the discharge capacity of the Niagara is greatly enlarged. In any event the lake can be held at substantially a fixed level only by greatly increasing the fluctuations of the discharge of the Niagara by an average of 50 000 cu. ft. per sec. each year and substantially reducing the ordinary low-water flow.

The question of the regulation of Lake Erie and of the Great Lakes, in general, was reviewed by the International Waterways Commission, a joint board of American and Canadian engineers. Its report, made in 1910, is a monumental study of the regimen of the Great Lakes. It pointed out the impracticability of regulating the outflow of Lake Erie so as to maintain a practically constant elevation of the lake, but presented a study of regulation based on the supplies of record, which was designed to hold Lake Erie within a range of 2.5 ft., as compared with the natural range, up to that time, of 3.8 ft. With the system proposed, there would have been an improvement of about 1 ft. in the ordinary low-water navigable depths on Lake Erie, about 0.3 ft. of which would have been due, however, to correcting the effects of diversions then existing. This improvement in the levels of Lake Erie would be at the cost of throttling the flow of the Niagara to its record minimum for a considerable part of the time, of increasing the fluctuations in Lake Ontario and in the St. Lawrence, and of lowering the minimum recorded level on Lake Ontario by about 4 in. In view of these counterbalancing injuries, the recommendation of the International Waterways Commission was adverse to the construction of regulating works in the Niagara River, or elsewhere.

OTHER PROPOSALS

Following this report various unofficial studies were made of the regulation of Lake Erie, some of which indicated that, with a suitable manipulation of the outflow, the disadvantages of the system studied by the International Waterways Commission could be overcome. The increasing demand for power, and the large amounts of power derivable from the head in the Niagara and St. Lawrence Rivers, pointed to the need for studying the manipulation of the levels of the lakes to develop the maximum useful storage, and the greatest steady outflow, in order to increase the potential output of primary power.

In 1919, Francis C. Shenehon, M. Am. Soc. C. E., made a report to the Chicago Sanitary District setting forth a scheme of regulation designed to equalize the summer flow of the Niagara River. As has been pointed out, the natural regimen affords an outflow which is ordinarily greatest in June and least in December. By retarding the flow after the danger of high water is past, it is possible to increase the autumn discharge. The results of the system studied indicated a gross gain of 19½ in. in the low levels of Lake Erie and of 12 in. in the high levels. The regulation, however, was based on an assumed diversion of 10 000 cu. ft. per sec. by the Chicago Sanitary District, and this diversion, with that of the Welland Canal and others, from Lake Erie would lower the lake by about 9 in. The effect of the proposed system of regulation would be, therefore, to raise the low levels by 10 in., and the mean levels by 3 in. more than could be effected by relatively inexpensive fixed works. The effect on the outflow of the Niagara is shown by the duration curves in Fig. 1.

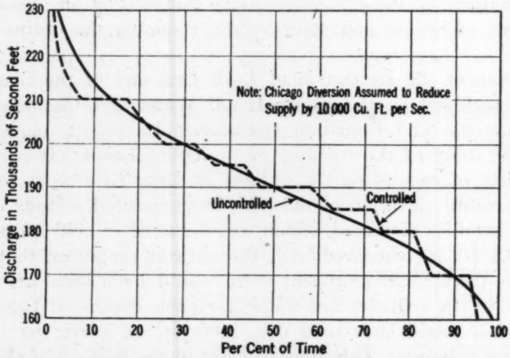


FIG. 1.—DISCHARGE DURATION CURVE FOR LAKE ERIE. SHENEHON PROPOSAL. PERIOD, 1893 TO 1902 AND 1907 TO 1916, INCLUSIVE.

In a paper[3] on the St. Lawrence Waterway before the Society, in 1924, Mr. Shenehon pointed out that the supply of water to the Great Lakes was sufficient to produce a steady outflow of 230 000 cu. ft. per sec., with a moderate annual hold-over storage. This is about 36 000 cu. ft. per sec. in excess of the present unrestricted minimum outflow.

In 1925 an Engineering Board of Review engaged by the Chicago Sanitary District prepared a scheme for the regulation of all the lakes, including Lake Erie. A careful review of the published computations and diagrams accompanying the report, and contained in a supplementary report of John R. Freeman, Past-President, Am. Soc. C. E., leads to the conclusion that the results of the system of regulation there considered would not differ substantially from those shown by the tentative study made by the Joint Board of Engineers on the St. Lawrence Waterway (presently to be discussed) so far

---

[3] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 444.

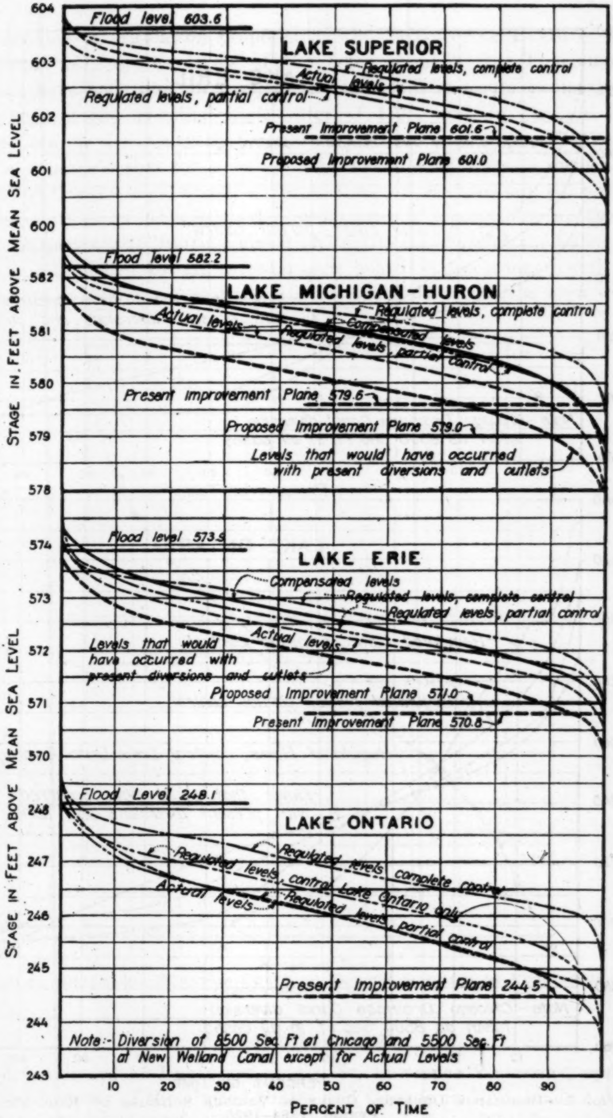


FIG. 2.—STAGE DURATION CURVE OF VARIOUS SCHEMES OF GREAT LAKES REGULATION. 1894–1925. NAVIGATION SEASON, APRIL 1 TO DECEMBER 1.

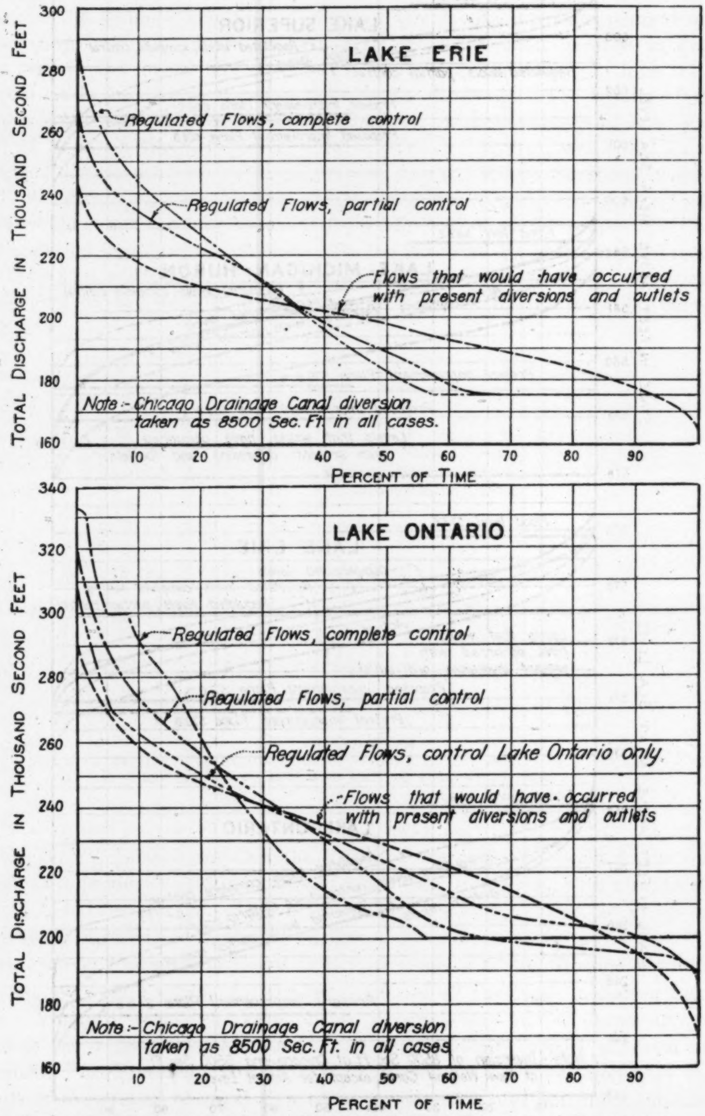


FIG. 3.—DISCHARGE DURATION CURVE OF VARIOUS SCHEMES OF REGULATION. PERIOD, 1894–1925.

at least as the fluctuations in the levels and discharges of the lakes are concerned.

The scheme of regulation proposed by the Engineering Board of Review contemplated raising the levels of the lakes about 2 ft. This suggestion appears to have been due, in part, to the fact that it would bring the high-water planes to the so-called high water of 1838, and, in part, to lack of available data on the injury that would result from such a rise in lake levels. It may be explained that the so-called high water of 1838 was a former datum plane on the Great Lakes, based, so far as some of the lakes are concerned, on incomplete data or no data as to the levels actually reached during that year. On Lake Superior the limiting elevation of the lake, under the rules for its regulation established by an international board in accordance with treaty requirements, is nearly a foot below the arbitrary datum of the high water of 1838.

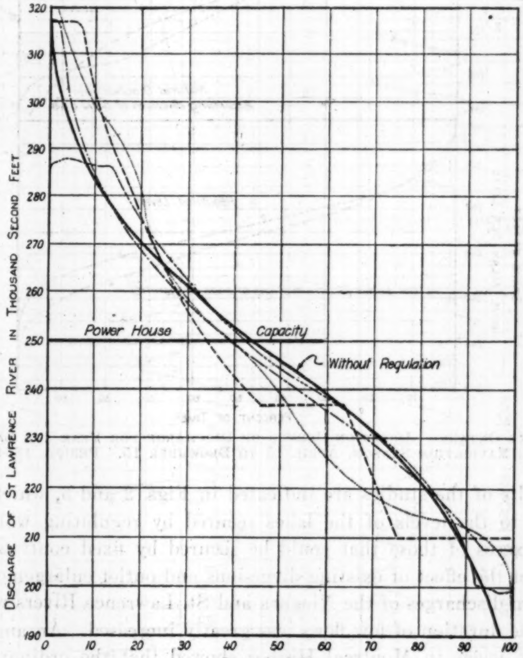


FIG. 4.—DISCHARGE DURATION CURVE OF VARIOUS SCHEMES OF REGULATION, LAKE ONTARIO ONLY.

ST. LAWRENCE BOARD

The Joint Board of Engineers on the St. Lawrence Waterway, appointed by the Governments of the United States and Canada in 1924, made a detailed study of the possibilities of regulating the Great Lakes. This included a full investigation into the permissible high levels of the lakes, the maximum and minimum discharges obtainable or permissible in the interconnecting rivers, and a revision of existing data on the water supply of the lakes since 1860, in the light of the latest evidence. With these data a tentative scheme of regulation, based on the budgeting of water between the lakes, was presented in detail. A modified program, based on a less comprehensive control of the outflow of Lake Huron, was also evolved.

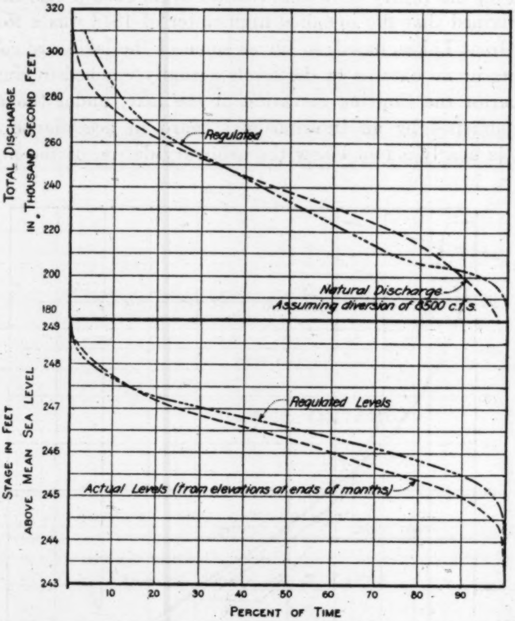


FIG. 5.—LAKE ONTARIO. DURATION CURVES OF DISCHARGE FOR YEAR AND LAKE LEVELS DURING NAVIGATION SEASON, APRIL 15 TO DECEMBER 15. PERIOD, 1861–1925.

The results of the studies are indicated in Figs. 2 and 3, which show that the benefits to the levels of the lakes secured by regulating works, are not greatly in excess of those that could be secured by fixed contractions which would correct the effect of existing diversions and outlet enlargements. While the minimum discharges of the Niagara and St. Lawrence Rivers were slightly increased, the duration of low flows was greatly increased. An analysis of the effect on the levels in Montreal Harbor showed that the ordinary low-water levels would be lowered by about a foot by the application of the program for comprehensive regulation. The benefits from regulation, when fully analyzed, were obviously such as to leave no ground for a recommendation that regulating works be constructed.

CONCLUSIONS

The regulation of Lake Erie may be directed to reducing the fluctuation in its levels, or to altering the natural fluctuation of its outflow. It appears

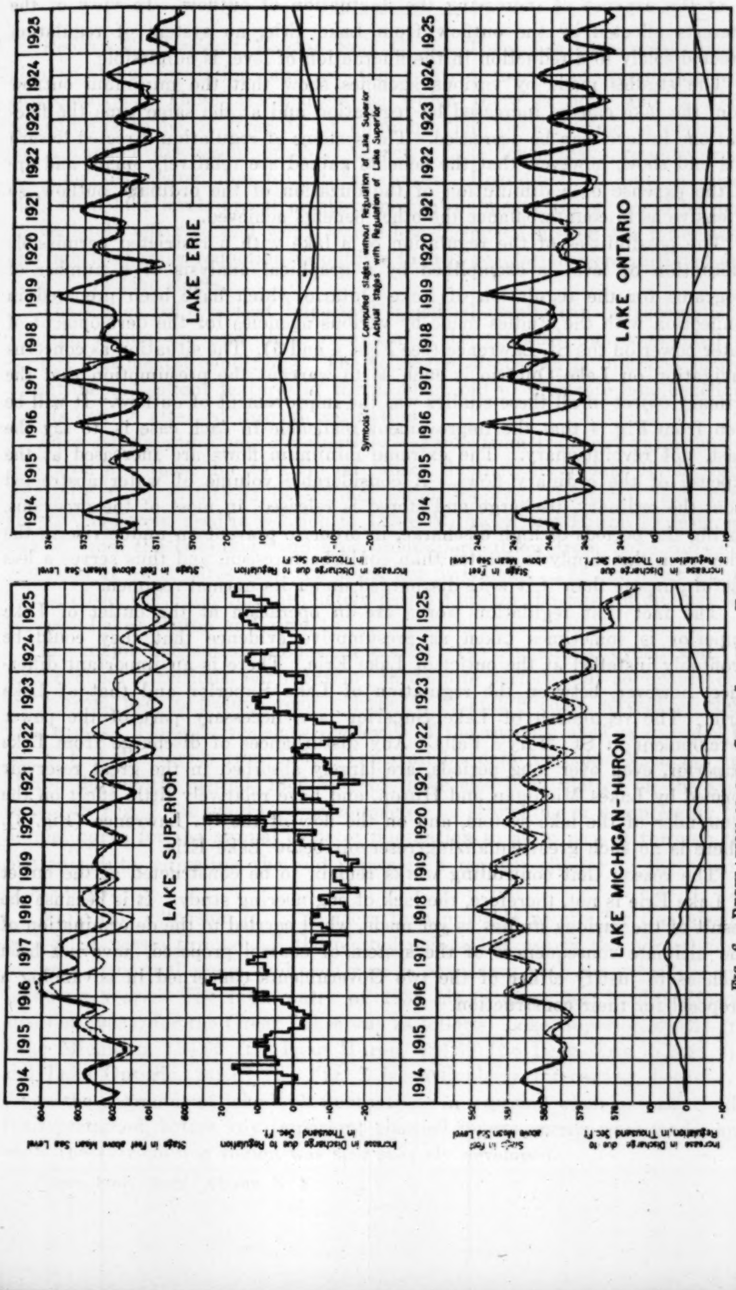


FIG. 6.—REGULATION OF THE GREAT LAKES. EFFECT OF PRESENT REGULATION OF LAKE SUPERIOR.

fundamental that a reduction in the fluctuation of the levels of a lake must be at the expense of increasing the fluctuation of outflow. In view of the interests affected by the outflow from Lake Erie, no system of regulation directed solely to a reduction in the fluctuation of level is admissible.

The studies made by various agencies show that the minimum outflow from the lake can be increased by regulation and at the same time the fluctuations in level can be decreased. The systems of regulation directed toward this end show, however, that the benefits gained are relatively small, and are at the expense of a diminution of the duration of the ordinary outflow, so extensive as to counterbalance the other benefits achieved.

The possibilities of the regulation of a lake with a restricted permissible fluctuation of level is exemplified by a graphical analysis of a number of programs for the regulation of Lake Ontario which have been prepared in connection with the studies made by various agencies for the development of water power on the St. Lawrence (see Figs. 4 and 5). The situation as concerns navigation on Lake Ontario is such as to warrant the presumption that the primary object in all these studies was the improvement of outflow. It will be seen from Fig. 4 that the improvement realizable in each case is, to say the least, not revolutionary. The extreme minimum flows are increased at the expense of the ordinary flows. A considerable volume of water abstracted from the ordinary discharge and stored is released, in most of the programs, during the periods of high discharge, in order to prevent an undue rise in the lake when the supply is greater than could be foreseen, and thus serves a less useful purpose than if it were discharged under its normal regimen.

The fact that regulation works are in operation at the outlet of Lake Superior is sometimes taken as presumptive evidence that they could be profitably installed at the outlet of Lake Erie. There is an important difference, however, between the regulation of Lake Superior and that of Lake Erie. The regulation of Lake Superior is a necessary part of the power development at St. Mary's Falls. Any disturbances of discharge from Lake Superior, even over long periods, are largely absorbed in the great reservoir formed by Lakes Michigan and Huron, and have relatively little effect on the remainder of the lake system, and on the flow of the St. Lawrence (Fig. 6). There is no such great equalizing reservoir below Lake Erie.

The reason that controlling works remain to be constructed at the outlet of Lake Erie is not, therefore, the lack of engineering study. It is because the result of the various studies as yet made, when carried to the determination of the ultimate consequences of the system of control proposed, have not been such as to justify either of the two Governments concerned in advancing a proposal for their construction.

DISCUSSION

Robert E. Horton,[4] M. Am. Soc. C. E.—There has been much discussion of lake regulation and of the relative merits of regulation and compensation. These are matters on which there is now, and is likely to continue to be, a diversity of opinion. The present time is opportune for a further discussion of this subject. Lake levels have recently risen to a marked degree due to natural causes. Regulation, if put in operation when lake levels are at or above normal or mean stages, would be more immediately effective and would cause less inconvenience on the lower lakes through temporary reduction in outflow than if regulation were initiated at a time of low lake stages.

The writer believes that important beneficial results can be accomplished through regulation, and these results may be attained unaccompanied by serious detrimental conditions. The following discussion is intended mainly to outline the salient features of the problem and to illustrate how beneficial results can be attained through regulation.

Lake Levels and Their Fluctuations.—Aside from transient variations in lake level and those due to artificial causes, fluctuations in lake levels result primarily from variations in the net water supply. The term, "net water supply", is herein used in relation to Lakes Michigan and Huron to designate run-off from the tributary land area, including outflow from Lake Superior, and yield of water surfaces of the lakes, less diversions. The net water supply is, therefore, the water that ultimately passes through the connecting channels between Lakes Huron and Erie.

The major fluctuations of lake levels are: (*a*) An annual cycle; and (*b*) long cyclical variations, not recurring in definite periods, but commonly covering from five to twenty years. Figs. 7, 8, and 9 show several of these long cycles. These curves show the levels of the various lakes as recorded by the U. S. Lake Survey from 1860. The available data for Lakes Michigan and Huron during the period, 1836 to 1859, have also been plotted on the same scale. In Fig. 7 the diagram was completed by stretching or compressing the normal curve for the annual cycle of fluctuation so as to pass it through the plotted points, adjusting the end points in a manner similar to the closing of a survey.

While this extension gives no more than probable results for the missing periods, it passes through all the plotted points, has nearly the normal form in years of normal range, and each year is consistent with the preceding and following years. The high water of 1838 above mean sea level (adjusted levels of 1903), for the Great Lakes was as follows: Lake Superior, 605.32 ft.; Lake Michigan, 584.69 ft.; Lake Huron, 584.69 ft.; Lake Erie, 575.11 ft.; and Lake Ontario, 248.98 ft. Fig. 7 is particularly interesting, because it shows the character of lake-level fluctuations in a period when natural conditions pertained, before any important channel improvements were made, and while the surrounding region was still sparsely developed.

---

[4] Cons. Hydr. Engr., Albany, N. Y.

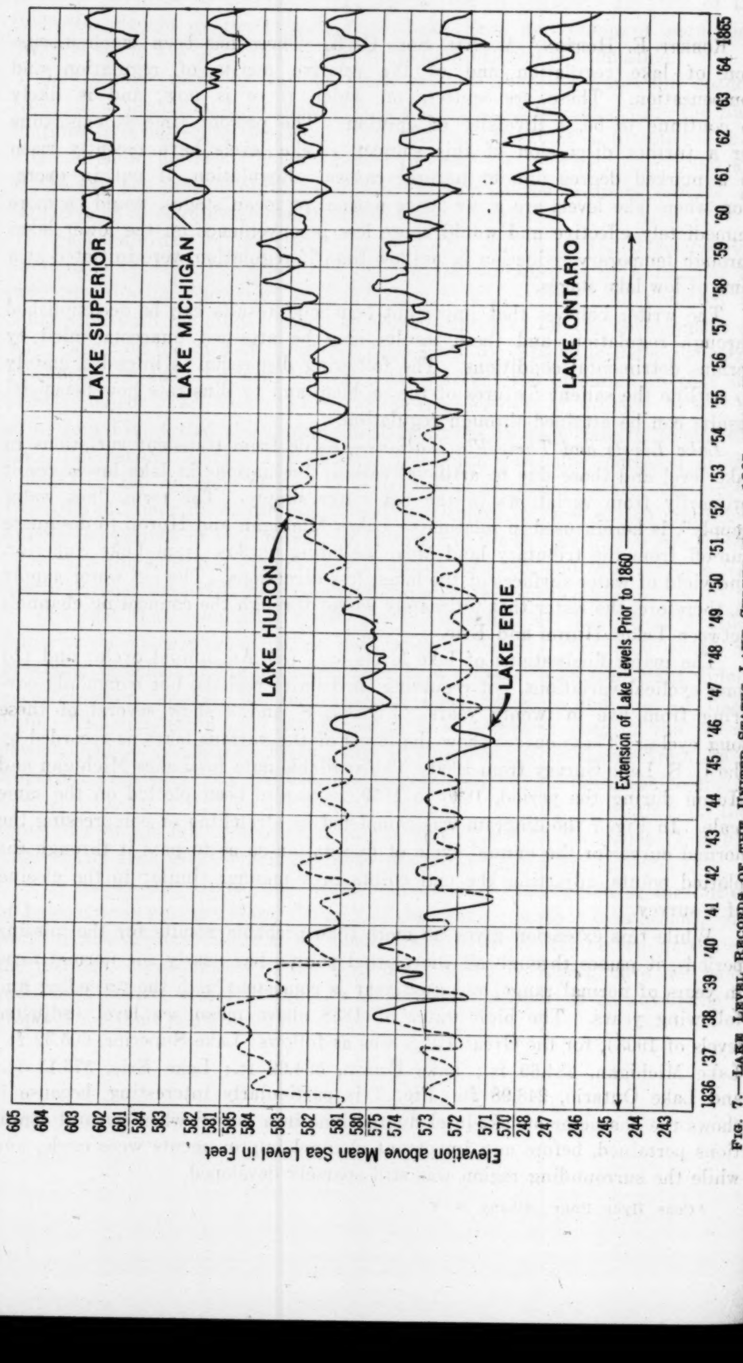


FIG. 7.—LAKE LEVEL RECORDS OF THE UNITED STATES LAKE SURVEY UP TO 1865, WITH EXTENSION BACKWARD FROM 1859 TO 1836.

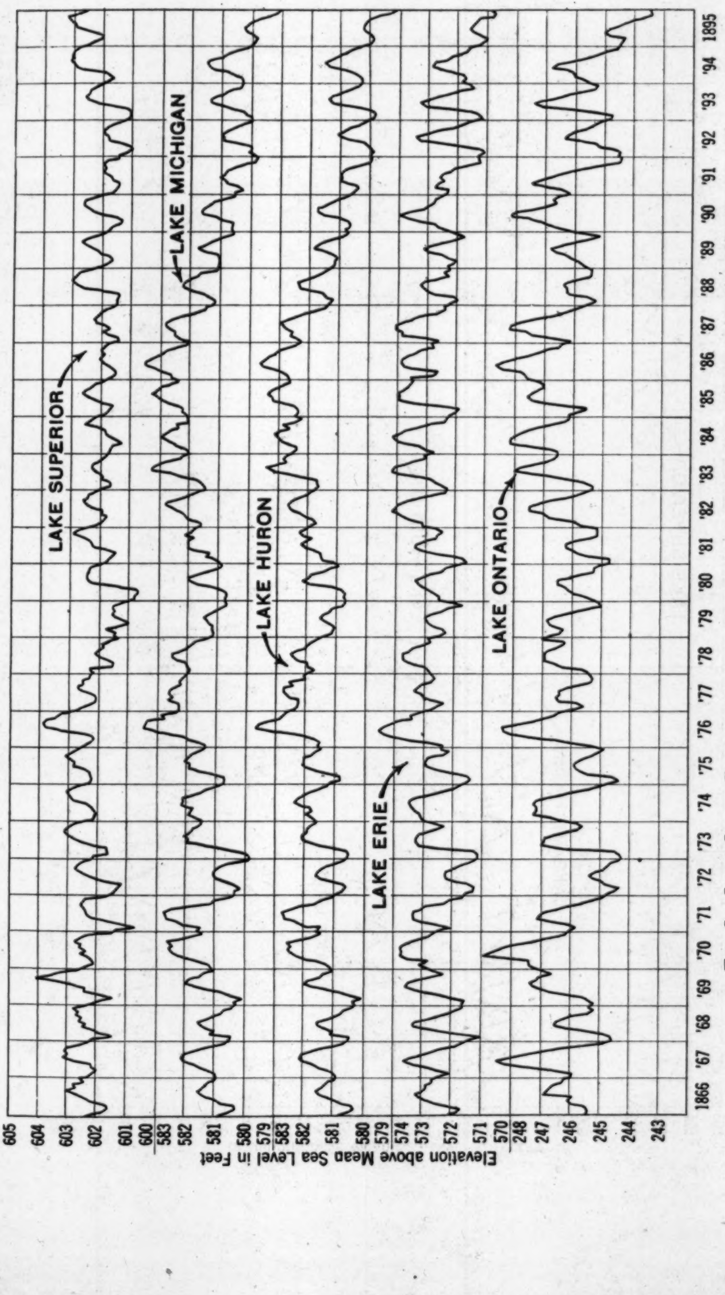


FIG. 8.—LAKE LEVEL RECORDS OF THE UNITED STATES LAKE SURVEY FROM 1866 TO 1895.

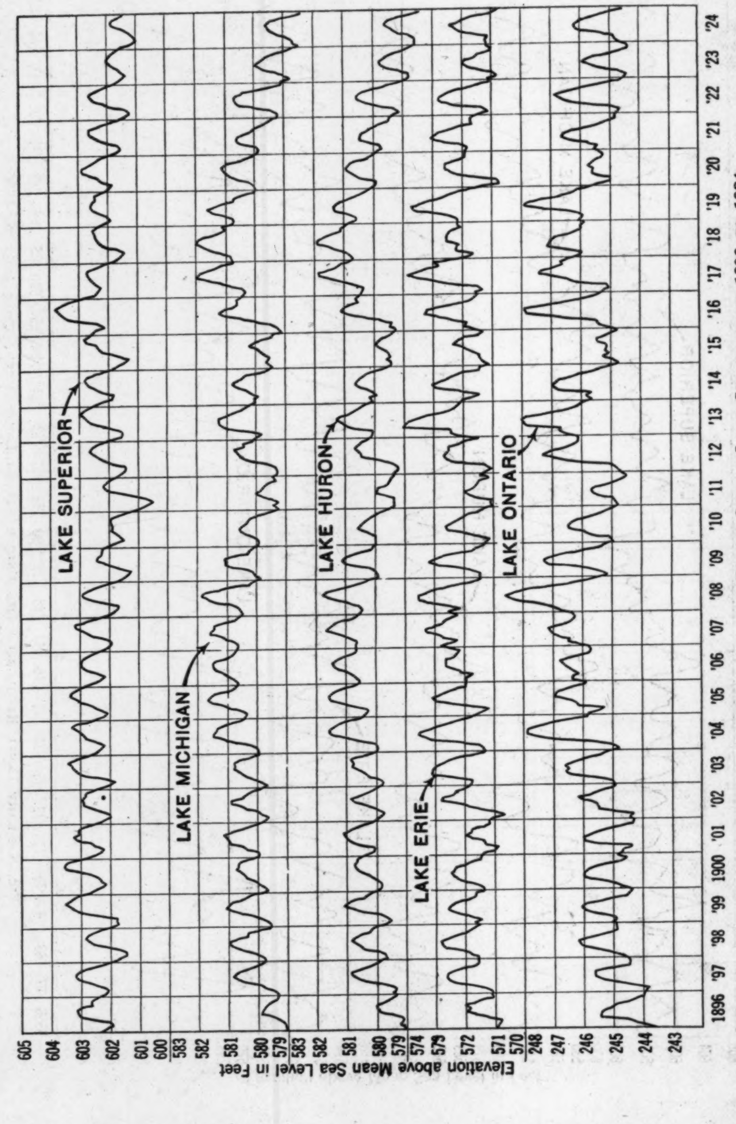


FIG. 9.—LAKE LEVEL RECORDS OF THE UNITED STATES LAKE SURVEY FROM 1896 TO 1924.

In 1847, Lake Huron fell to a level nearly as low as any in recent years. In 1852, the level of Lake Huron was almost as high as at any subsequent date. Two cycles of fluctuation are shown prior to 1860, similar in character to cycles which have subsequently occurred. That the lakes were higher in 1838 than at any time since seems certain. The elevation of the high-water plane of 1838 probably is not very accurately established.

Evaporation from the Great Lakes appears to be relatively more constant than rainfall. Since variation in rainfall is the principal cause of natural variation in lake levels over long periods, it is interesting to note that the irregular periodic character of long-cycle variations in lake levels is very similar to that for rainfall, as shown by the longest available records.

It is evident, therefore, that, unless there is a permanent change in rainfall, lake levels will tend to fluctuate in the future much as they have in the past, and the important conclusion may immediately be drawn that, aside from permanent changes due to artificial causes, the levels of the lakes, if left to themselves, will ultimately return from their present (1928) low stages to normal and even to extreme high stages, such, for example, as persisted in the Eighties.[5]

It may be noted that in the Eighties (see Fig. 8), Lake Michigan reached a maximum stage of 583 ft., or more, for four successive years, and during three of these years the lake remained above Elevation 583 for three or four months in each year.

The mean stage of Lakes Michigan and Huron in the maximum month of each of the years, 1924 to 1927, was about 579, while for the four years, 1883 to 1886, the mean of the maximum month was above 583, or more than 4 ft. higher than in the recent period. Similar relations hold for mean and minimum stages. Special attention is called to this matter because of its application to lake regulation and to the encroachment of structures on the lake margins. It appears certain that structures adjoining the lakes, placed at too low elevations in the belief that lake stages would remain substantially as at present, are bound to suffer serious consequences.

Of course, lake stages approaching those of the late Fifties or Eighties may not be realized fully because of the present lowering of lake levels as a result of diversions and channel improvements. These improvements have had a combined effect in lowering the mean levels of Lakes Michigan and Huron by an amount variously estimated as between the limits of 1 ft., or less, and 1½ ft., or more.

Lake Regulations.—Lake carriers commonly utilize the full available draft depth. Tonnage must be provided adequate to carry the commerce offered in periods of minimum as well as of maximum lake level.

Various desiderata of lake-level control include:

(*a*) Increase of the minimum depth in harbors and connecting channels.

(*b*) Reduction in fluctuation in lake levels, or the maintenance of a more equable navigable depth.

(*c*) Restoration of the depth lost in harbors and connecting channels through the combined effect of diversions and channel deepening.

---

[5] This statement (written in July, 1928), is confirmed by the upper lake stages of the summer of 1929.

There are three methods by which increased navigable depths may be obtained:

1.—By dredging the harbors and deepening the connecting channels.
2.—By compensation or fixed control in the outlet channels without regulation.
3.—By the construction of regulating works on the lake outlets.

The question may be asked: If increased navigable depth in connecting channels and harbors is required, why not provide it by dredging and excavation, as in the past? Other things remaining unchanged, dredging of course will provide increased depth only in the harbors dredged, while deepening of outlet channels—as already learned by experience—unless fully compensated, provides deeper water in the improved channel at the expense of lower levels in the tributary lake and, consequently, reduces the navigable depth in all harbors on the lake, the outlet of which is deepened.

Compensation consists in putting material back into an outlet channel outside the deepened navigable fairway with a view to preventing the increase in discharging capacity which would otherwise result from channel deepening. It amounts approximately to maintaining a constant cross-section, where the channel is deepened, by reducing the effective width sufficiently to offset the increased depth.

Fixed control structures are for the same purpose and operate in the same manner as compensation. Compensation in part has been carried out in the past in connection with improvement of the connecting channels between Lakes Huron and Erie.

Compensation seems very simple and effective, but it is subject to some serious defects as a remedy for reduced lake levels. Take, for illustration, a case such as that of the St. Clair River, where the discharge bears nearly a simple linear relation to the stage of the lake for moderate variations in lake level. Let it be assumed that compensation is made with reference to maintaining unchanged the mean stage at the outlet channel after the channel is deepened, and that this is accomplished by compensation outside the navigable waterway in such a manner that the relation of stage to discharge still remains linear. In Fig. 10 the line, *AB,* is the elevation of the original outlet control sill. The original relation between water surface elevation and discharge is shown by the line, *AH.* After deepening the channel to the elevation of the line, *DJ,* and reducing its width, the stage-discharge relation is as shown by the line, *DE.* The two discharge curves intersect at the point, *F,* corresponding to the mean stage, *M,* which is unchanged.

Table 1 is included to interpret the lines in Fig. 10. It shows that, while the navigable depth of the channel is increased at all stages, the range of fluctuation of lake levels is increased so that the maximum lake stages will be higher and the minimum stages lower than before the channel improvement takes place. In other words, the minimum navigable depth in all harbors on the tributary lake will be less than before the improvement, although the minimum depth in the outlet channel will be increased. This is due to the fact that, as a result of the compensation, the water level through the improved channel is not reduced as much as the channel itself is deepened.

Of course, compensation need not be made with reference to the mean stage. It can be made, for example, to preserve the minimum stage of the lake unchanged. In that case, however, there will be a larger increase in maximum stages than in the example just given. In general, channel improvement with compensation or fixed-outlet control alone will be found to operate to advantage in some respects and to a disadvantage in others. This is because compensation can only maintain the channel-discharging capacity unchanged at one particular stage.

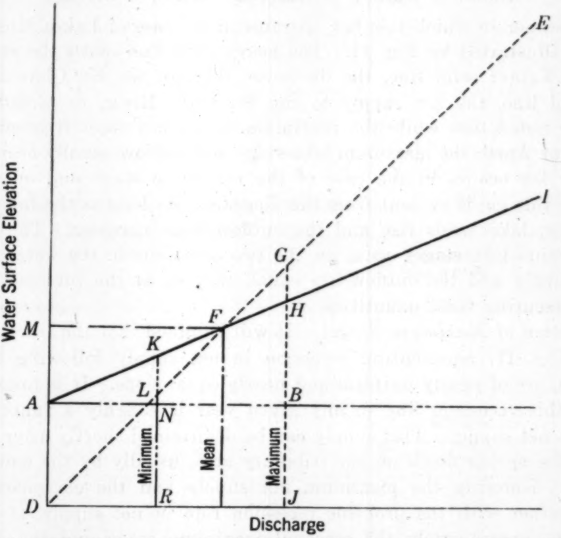


FIG. 10.—EFFECT OF CHANNEL DEEPENING AT LAKE OUTLET CONTROL POINT WITH "COMPENSATION" THROUGH REDUCTION IN WIDTH SO AS TO MAINTAIN PERMANENT MEAN LAKE STAGE UNCHANGED.

While a certain increase in maximum lake stages in the lower lakes over those pertaining in recent years should and undoubtedly can be tolerated, a decrease rather than an increase in range of fluctuation of the lake levels is desirable. Only thereby can the mean and minimum lake stages and navigable depth be increased without the certainty that at some future time the lakes will rise to objectionable heights.

TABLE 1.—EFFECT OF IMPROVEMENTS ON LAKE LEVELS.

Lake level.	Before improvement.	After improvement.	Increase, or decrease.
Mean stage	*M*	*M*	....
Stage at maximum discharge	*H*	*G*	Increase, *GH*
Depth at maximum discharge	*HB*	*GJ*	Increased
Stage at minimum discharge	*K*	*L*	Decrease, *KL*
Depth at minimum discharge	*KN*	*LR*	Increased

In contradistinction to compensation, or simple control, lake regulation consists in placing permanent structures below its outlet so that the water can be held back or released at will and both the lake levels and outflow can be controlled artificially to some degree. The ideal of constant lake levels at a moderately high stage with constant outflow rate is obviously unattainable.

The regimen of lake levels and outflow is controlled by the operation of the general law that applies to storage of all kinds, as follows:

Inflow = Outflow + Gain, or — Loss, of Storage

The manner in which this law operates in the case of Lakes Michigan and Huron is illustrated by Fig. 11. The heavy solid line shows the stage of the lakes, the lighter solid line, the discharge through the St. Clair River, and the dashed line, the net supply to the St. Clair River, as already defined. It will be noted that while the maximum net supply most frequently occurs in March or April, the maximum lake stage and outflow usually occur in July. A similar lag occurs in the case of the minimum stage and outflow. The reason for this lag is evident from the diagram. As long as the inflow exceeds the outflow, lake levels rise and the outflow rate increases. The maximum and minimum lake stages occur on the two occasions in the water-year when the net supply and the outflow are equal, that is, at the intersection of the lines representing these quantities.

Prediction of Maximum Stages.—It will be noted that the lines, *AA′*, *BB′*, etc., on Fig. 11, representing recession in net supply following the spring maximum, are of nearly uniform and nearly equal slope. It is found that the slope of this recession line in any given year is roughly a function of the maximum net supply. This supply can be determined shortly after the occurrence of the spring floods on the tributary area, usually by the end of March or April. Knowing the maximum net supply and the corresponding lake stage, together with the probable recession rate in net supply, it is possible to predict approximately the seasonal maximum stage and the date of its occurrence from two to four months in advance. This fact may prove to be of value in the operation of any plan of lake regulation, since it makes it possible to determine in advance when an abnormally high maximum stage is likely to occur. It is then also possible to begin at least two months in advance to slice off the crest of the maximum by discharging the excess water. These steps facilitate the operation of regulating works in such manner as to increase the minimum stages of the lakes without either increasing the maximum stage or reducing the outflow during the minimum period.

The Use of the Storage Schedule.—In the utilization of storage reservoirs in conjunction with power developments it is commonly desired to maintain a flow at all times at least equal to a predetermined minimum for the production of firm power. This result alone can be accomplished by keeping the reservoir always as nearly full as the draft requirements will permit. This method of operation is, however, generally wasteful of water. In addition to firm power, it is also often desirable to produce secondary power and to keep as much reservoir capacity available for flood control as power requirements permit.

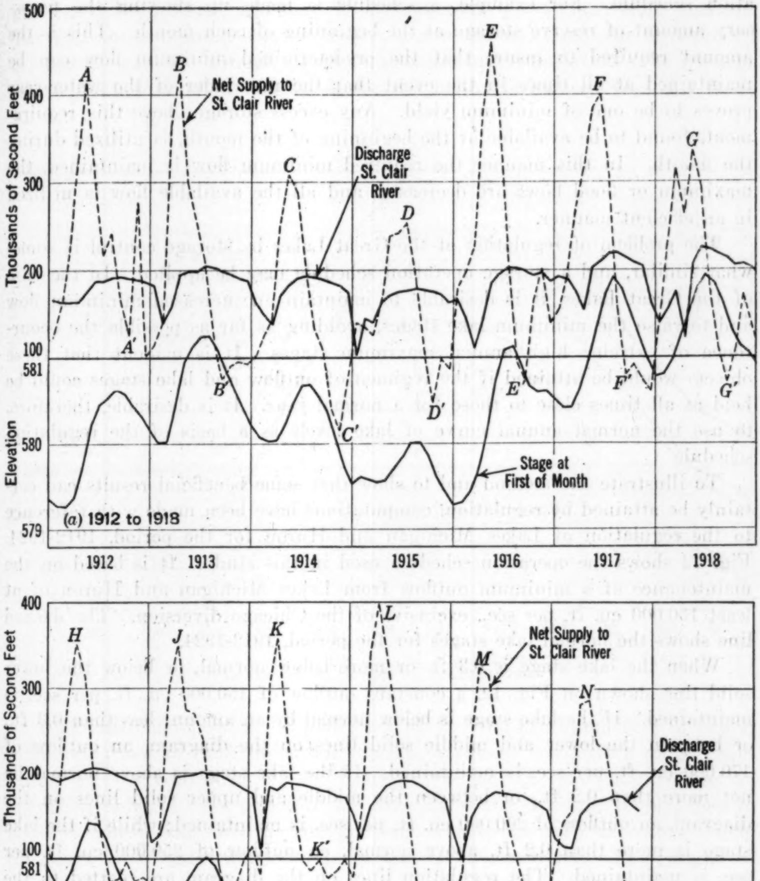


FIG. 11.—RELATION BETWEEN NET SUPPLY TO ST. CLAIR RIVER AND MAXIMUM AND MINIMUM ANNUAL STAGE AND DISCHARGE OF LAKES MICHIGAN AND HURON.

These combined functions are best served by the use of a storage operation schedule. For example, a schedule is made up showing the necessary amount of reserve storage at the beginning of each month. This is the amount required to insure that the predetermined minimum flow can be maintained at all times in the event that the remainder of the water-year proves to be one of minimum yield. Any excess storage above this requirement, found to be available at the beginning of the month, is utilized during the month. In this manner the required minimum flow is maintained, the maximum or flood flows are decreased, and all the available flow is utilized in an efficient manner.

The problem of regulation of the Great Lakes by storage control is somewhat similar, and a storage operation schedule may be applied. In the case of the Great Lakes, it is desirable to maintain an increased minimum flow and to raise the minimum lake stages, avoiding as far as possible the occurrence of extreme high annual maximum stages. It is evident that these objects would be attained if the regimen of outflow and lake stages could be held at all times close to those for a normal year. It is desirable, therefore, to use the normal annual curve of lake levels as a basis of the regulation schedule.

To illustrate the method and to show that some beneficial results can certainly be attained by regulation, computations have been made with reference to the regulation of Lakes Michigan and Huron for the period, 1912-1924. Fig. 12 shows the operation schedule used in this study. It is based on the maintenance of a minimum outflow from Lakes Michigan and Huron of at least 150 000 cu. ft. per sec., exclusive of the Chicago diversion. The dashed line shows the normal lake stages for the period, 1912-1924.

When the lake stage is 0.3 ft. or more below normal, or below the lower solid line shown on Fig. 12, a constant outflow of 150 000 cu. ft. per sec. is maintained. If the lake stage is below normal by an amount less than 0.3 ft., or between the lower and middle solid lines on the diagram, an outflow of 170 000 cu. ft. per sec. is maintained. If the lake stage is above normal by not more than 0.2 ft., or between the middle and upper solid lines on the diagram, an outflow of 200 000 cu. ft. per sec. is maintained; while if the lake stage is more than 0.2 ft. above normal, an outflow of 250 000 cu. ft. per sec. is maintained. The regulation lines on the diagram are plotted to the nearest 0.1 ft. for ease in operation and, therefore, are not quite parallel to the curve of normal stages.

In determining the results of operation in accordance with this schedule it is assumed that the operator did not have any advance information as to lake stages to be expected nor as to amount of outflow to be released from the Lake Superior regulating works during subsequent months. The regulation is based wholly on information available to the operator on the first of the given month.

Fig. 13 shows the results of operation by this method. The solid line in Fig. 13(*b*) and Fig. 13(*e*) shows the actual lake stages; the lower dashed line shows the regulated stages if the regulation had begun in January, 1912, without any attempt to boost the mean stage. It will be noted that the extreme minimum stage in the period covered is raised 0.35 ft. above the unregulated minimum, and that the annual minima are much more uniform. The extreme maximum stage is 0.65 ft. below the maximum for unregulated conditions, and the annual maxima are much more uniform. The extreme range of fluctuation of lake levels for the period is reduced from 3.3 ft. to 2.3 ft.

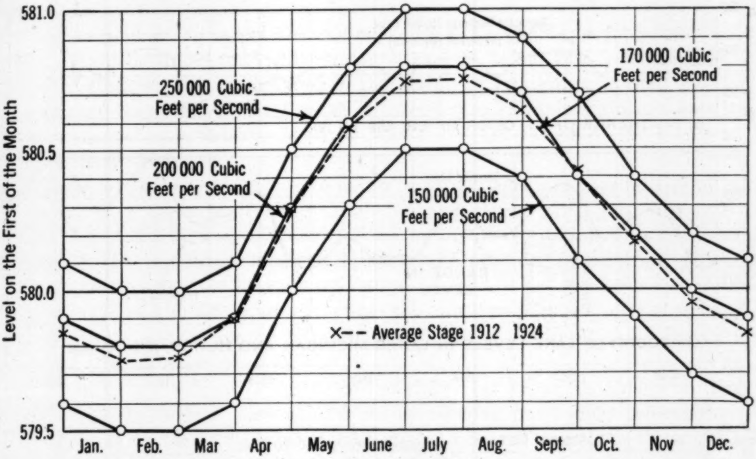


FIG. 12.—OUTFLOW TO BE RELEASED THROUGH ST. CLAIR RIVER FOR GIVEN STAGE OF THE LAKES ON THE FIRST OF EACH MONTH.

It is possible, of course, to hold back part of the maximum flow in one or more years and to raise the mean stage to a chosen level, after which regulation can be continued on the basis of the new normal stage curve. If operation is begun in a period of low lake levels and deficient supply, then to boost the mean stage will inevitably result in a temporarily decreased supply to Lakes Erie and Ontario and the St. Lawrence River. This might be objectionable and can be avoided by waiting until the lake levels rise naturally to a suitable stage and then beginning regulation on the basis of the new mean level, which thereafter will be maintained as closely as possible.

Regulation of Lakes Michigan and Huron on the basis of an annual mean stage of 1 ft. above that of 1912, could begin in this manner in 1918 and thereafter would provide stages as shown by the upper dashed line on Fig. 13(*b*) and Fig. 13(*e*). The maximum stage is not raised above the unregulated maximum at the beginning of operation in 1918, but the mean and extreme minimum stages for the period, 1918 to 1924, are both raised about 1 ft., and similar results can be obtained in subsequent years.

Fig. 13(*a*) and Fig. 13(*d*) show the discharge. The solid line is without, and the dashed line with, regulation. It will be noted that the minimum outflow is increased materially for a period of several months in nearly every year. Two objections may be made to regulation in the manner described:

1.—That the full apparent increase in minimum discharge may not be realized because of ice obstruction in the outlet channel.

2.—That the maximum outflow rates are increased materially at times over those pertaining for the same lake stages with unregulated outflow.

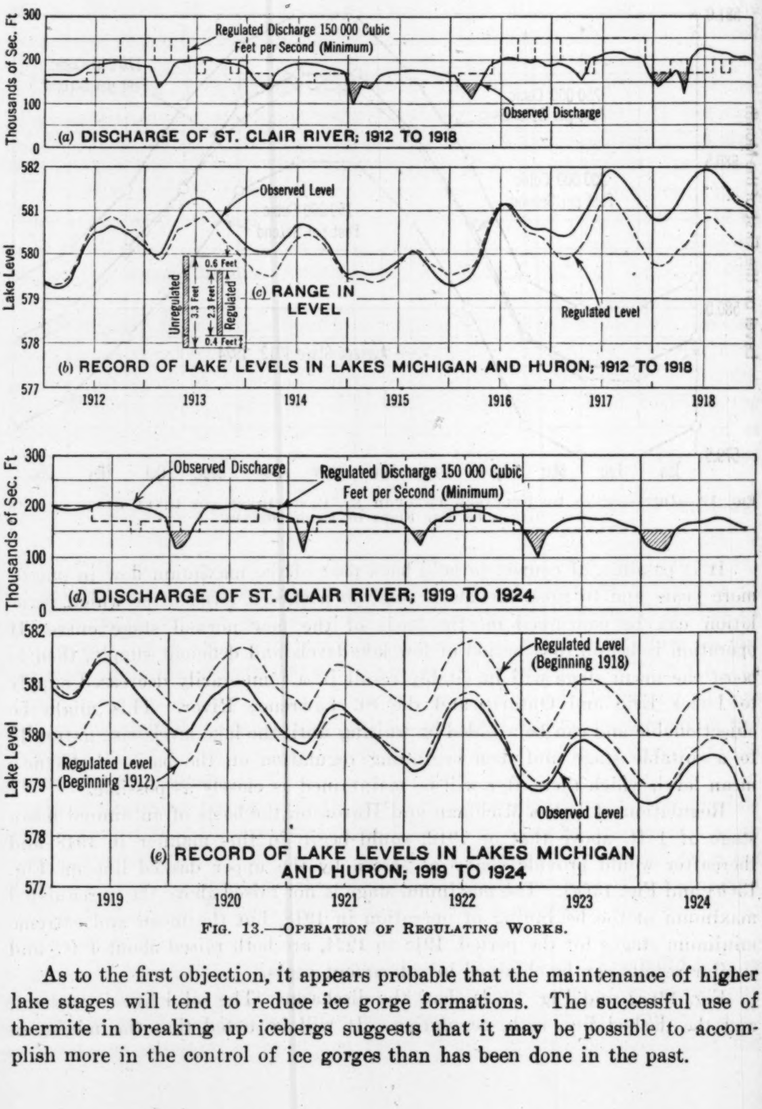


FIG. 13.—OPERATION OF REGULATING WORKS.

As to the first objection, it appears probable that the maintenance of higher lake stages will tend to reduce ice gorge formations. The successful use of thermite in breaking up icebergs suggests that it may be possible to accomplish more in the control of ice gorges than has been done in the past.

It is true that more water is discharged at times than would be discharged at the same lake stages with unregulated outflow. This condition undoubtedly could be mitigated to some extent by closer regulation, say, on a weekly instead of a monthly basis, by the use of maximum stage predictions made two or three months in advance, in some manner similar to that already described, and by the use of advance information as to the quantity of water expected to be released from the regulating works on the outlet of Lake Superior.

Some increase in outlet channel capacity will probably be found necessary, or at least desirable, and this factor should be taken into consideration in further study of the subject and in the design of regulating works.

It may be repeated that Fig. 13 is intended to be mainly illustrative; it is not presented as a definite basis of regulation, but rather as the result of a preliminary study.

Much of the discussion herein given as to methods and possibilities of lake regulation in the case of Lakes Michigan and Huron applies equally well to Lake Erie. Regulation of Lake Erie by means of regulating works above the crest of the rapids at Niagara Falls would provide, in addition to other advantages, means for diurnal regulation of outflow for power use as well as temporarily withholding or diverting the outflow in such a manner as to facilitate the restoration of Niagara Falls and the amelioration or prevention of future destructive erosion.

Finally, this subject of regulation of the Great Lakes should by no means be dismissed. A final plan should not be adopted until more information is available as to feasible limits of lake stages, or until more careful study has been given to the hydrologic and hydraulic aspects of the subject as well as its commercial and navigational aspects.

Lynn L. Davis,[6] **M. Am. Soc. C. E.**—The author has presented a most excellent summary of the various studies that have been made on this subject, and has very ably set forth the difficulties attendant on the solution of the problem. The speaker desires to give some facts concerning the importance of the commerce on the Great Lakes, and of the growing commerce on the Niagara River, because he considers that commerce is entitled to first consideration in a study of lake regulation. A few facts relating to the hydraulics of the Niagara River are also included.

Commerce of the Great Lakes.—The general public and most engineers have little conception of the extent of this industry and its tremendous and far-reaching effect on the life of the Nation. The latest published statistics of the commerce of the United States is for the season of 1926, and they show that the Great Lakes tonnage comprised more than 25% of all the water-borne tonnage handled in and on the United States ports and waterways. It was three times the Pacific Coast traffic; two and one-half times the Gulf Coast traffic; and more than two-thirds the Atlantic Coast traffic. It was larger than the tonnage handled on all the rivers and other inland waterways in the United States. Its relative importance to all other inland waterway traffic is not correctly represented by the tonnage because the length of haul is much greater on the lakes. Of a total ton-mileage of more than 99 500 000 000 for all inland traffic, that on the Great Lakes consisted of more than 90 000 000 000 and all others of 9 500 000 000, or a ratio of nearly 10 to 1. This does not include the foreign traffic on the lakes, which constitutes more than 12% of the total lake traffic. These inland waterways consist of the rivers, canals, and connecting channels and the Great Lakes System.

---

[6] Prin. Civ. Engr., U. S. Engr. Office, Buffalo, N. Y.

The bulk of the freight carried on the Great Lakes consists of articles fundamental to the industrial life of the Nation and of foodstuffs for man and beast. In 1926, about 372 000 000 bushels of grain, 10 000 000 bbl. of flour, 60 000 000 tons of iron ore, and 14 000 000 tons of coal, passed through the Sault Ste. Marie Canals. Of the total freight tonnage carried on the Great Lakes, iron ore and its manufactured products constituted about 50%; coal, 26%; stone, 11%; grain and its products, 5½%; sand and gravel, 3%; and miscellaneous, 4½ per cent. About 98% of the iron ore handled in the ports of the United States is carried on the Great Lakes.

Duluth-Superior (Minn.-Wis.), Harbor, situated at the head of Lake Superior, is the second largest port in the United States. It received and shipped more than 53 000 000 tons of freight in 1926, which is surpassed only by New York, N. Y. The third port, Philadelphia, Pa., had a tonnage only a little more than one-half that of Duluth.

As has been stated by Colonel Pillsbury, the navigation interests are highly organized and will take advantage of every available inch of draft. It has been estimated that every such additional inch is worth $500 000 per season to the lake shippers. It is apparent, therefore, that a system of regulation that would increase the available depth 6 in. would be worth $3 000 000, and it is reasonable to assume that a large part of this saving would accrue to the producer and consumer.

Commerce on the Niagara River.—The average tonnage of freight carried on the Niagara River for the years 1924, 1925, and 1926, was about 3 333 000, which is about double what it was in 1914, when the Black Rock Ship Canal was opened to traffic. This ship canal is about 3.5 miles long and was built to provide safe navigation for the largest lake freighter around the very swift and shallow rapids at the head of the Niagara River. It displaced the section of the old Erie Canal that skirted the east bank of the river at this point, and now carries all the Buffalo traffic of the New York State Barge Canal. The present traffic in the Black Rock Ship Canal consists of nearly 50% Barge Canal traffic, which requires only 12 ft. of water. Less than 25% of the traffic utilizes the full available depth of about 19 ft.

The development of the deep-water pool in the river between Buffalo and Tonawanda, N. Y., as additional harbor space for Buffalo has been advocated, and such improvement has many points of merit. If the Black Rock Ship Canal were widened to not less than 300 ft. in the canal section and 400 ft. in open river to the full depth of the other connecting channel in the Great Lakes System, and if the mainland and Grand Island sides of this reach of the river were provided with adequate railroad service, the speaker believes that this frontage would develop rapidly.

It would be especially adapted to the winter grain-storage business which has recently developed in Buffalo Harbor. It is not unusual to have from fifty to one hundred of the largest freighters on the lakes storing grain in Buffalo Harbor during the winter. The grain is unloaded as required for rail shipment to Atlantic Coast harbors or to mills. This necessitates the service of several powerful ice-breaking tugs during the entire winter to move these vessels to and from the elevators. If this business could be transferred to the Niagara River there would be little interference from ice, because the river never freezes over and there is usually little floating ice from Lake Erie, except during from four to five weeks in the spring when the lake ice is broken up, but by that time most of the grain has been unloaded.

The prospects of the extension of the deep-water channel from Tonawanda to the Falls in the near future is not bright, because of the extremely high cost of the work due to the prevalence of solid rock. Such a channel would also require compensating works to prevent a considerable lowering of the river and lake. The cost of such an extension would be greater than all that has been spent to date by the Federal Government on Buffalo Harbor and Niagara River, and the most favorable estimates of probable future tonnage through such a waterway is less than 3% of the present tonnage in Buffalo Harbor.

In view, therefore, of the vast investment of money and effort in the navigation industry on the Great Lakes and the rapidly growing importance of the Niagara River commerce, their interests must enter in a very large measure into the final determination of method and amount of regulation or compensation.

Hydraulics of the Niagara River.—The speaker does not know of any data giving the levels of Lake Erie or Niagara River previous to the placing of artificial works in the head of the river in 1823-25, when the Erie Canal was built. This work consisted of closing the part of the river that flowed between Squaw Island and the mainland. Three structures were built: A dam; a dike along the east bank of that island from the dam to the head of the island; and a pier, which is virtually a dam, from thence to the head of the river, a distance of about 2½ miles, terminating on Bird Island, a rocky reef that was and is submerged during periods of high water. It was subsequently extended easterly about 3 100 ft. to increase the size of the harbor.

No data are available to determine how much this restriction raised the level of the water in Lake Erie, but it must have been at least 0.3 ft. and it may have been as much as 1.0 ft.

The original improvement of the Niagara River for lake navigation consisted of cutting a channel 400 ft. wide and 16 ft. deep through the reef at the head of the river. The cut consisted principally of the removal of portions of shoal spots on the reef and, while it must have caused a slight lowering, the total was probably not more than a fraction of an inch. Recent channel improvements between Black Rock Lock and Tonawanda have undoubtedly also resulted in a slight lowering of Lake Erie, but the sum total lake lowering from all channel improvements in Niagara River is less than 1 in.

For many years sand and gravel have been dredged from the river for commercial use. Most of this material was removed from a section about 4 miles long, extending down stream from the head of Squaw Island, or from 3 to 7 miles from the head of the river. No record of the total quantity dredged is available, but it must have been more than 10 000 000 cu. yd. About one-fourth of Squaw Island and three-fourths of Strawberry Island have been removed.

From studies of the effect of this dredging, made in 1926, it was estimated that the lake was lowered 0.3 ft. and the river at the foot of Squaw Island, 0.5 ft. All permits for dredging in the Niagara River were canceled as a result of these studies, and no such dredging has been allowed in the river since August, 1926.

The effect of all the diversions and of artificial changes in the Niagara River, on the level of Lake Erie, is shown in Table 2.

The levels in the river would not be affected in the same manner by the diversions listed in Table 2. All diversions made above the head of the river would affect the level in the Grass Island Pool at Niagara Falls slightly less than 50% as much as Lake Erie. For instance, the first two diversions in Table 2 would lower that pool only about 0.3 ft.; the third would have no appreciable effect because all the water is returned to the river; the fourth would have about the same effect on the pool as on Lake Erie; the fifth would lower the pool seven or eight times as much as the lake; and the sixth, the gravel dredging, would affect the pool level only slightly.

TABLE 2.—LOWERING OF LAKE ERIE DUE TO ARTIFICIAL CHANGES.

Diversion of water by:	Lowering, in feet.
Chicago Drainage Canal	0.41
Welland Canal	0.21
Black Rock Canal	0.03
New York State Barge Canal	0.01
Niagara Falls Power Companies	0.06
Sand-gravel excavation, Niagara River	0.03
	1.02
Probable compensation of Erie Canal structures built previous to 1825	0.32
Net lowering	0.70

It seems, therefore, that Nature has provided a very efficient regulating device in the natural channel of the Niagara River, whereby only about one-half the extreme fluctuations in Lake Erie are transmitted to the river at the Falls, or, with a rise in Lake Erie of 12 ft., which occasionally occurs, a corresponding rise of not more than 6 ft. takes place at the Falls. On the other hand, the frequent daily fluctuations in the level of the Grass Island Pool, due to the variation in the quantity of water used by the power companies, causes very little change in the current or river levels near the head of the Niagara River.

The studies made of lake and river regulation have thus far failed to reveal any safe method of improving on Nature; therefore, it seems that only such compensating works as are necessary to restore original conditions are justified at present.

The plan suggested by the St. Lawrence Waterway Joint Board of Engineers in its report dated November 16, 1926, seems entirely feasible; it is comparatively inexpensive and can be built gradually, observing the effect and modifying it until the desired results can be obtained.

This plan consists of the following:

(*a*) Four submerged weirs or sills extending across the deep portion of the waterway near its head, with their tops 13 ft. below the water surface, thereby forming no obstruction to shallow-draft vessels.

(*b*) A wall or dike extending parallel with the direction of the current in the river, near the westerly end of the submerged sills, with its top 4 ft. above water level.

(*c*) A weir, extending diagonally from the up-stream end of the wall or dike to the Canadian bank of the river, with its top at low water.

This diagonal weir would extend through comparatively shallow water and is designed to restrict materially the low-water discharge of the river, while permitting the full-width discharge at flood stages. It is estimated that these works would raise the level of Lake Erie about 0.7 ft., without greatly affecting the flow of the river after the new level in the lake is established.

It may also be found necessary to build compensating works in the river at or near the Grass Island Pool to balance the slightly less low-water flow that would result from the works constructed at the head of the river.

THERON M. RIPLEY,[7] M. AM. SOC. C. E.—This matter of lake regulation is based on the need of providing better water transportation facilities. Studies were started at least thirty-six years ago. The first report in the speaker's possession was issued in 1894, and numerous reports have been made since that time. The transportation interests are justified in demanding that something be done, even if it is no more than an admission that the engineers of the United States can give them no help in this matter.

The fact can be stated that compensation or regulation, or a combination of the two, can be made so as to help lake transportation, and the engineers and shippers along the Great Lakes should be able to create a public sentiment that would demand physical action and institute construction of the necessary works.

It is granted that the control of a storage reservoir having an area of more than 55 000 sq. miles, which is fed not only by its own water-shed, but by that of another reservoir containing 32 000 sq. miles, contains hydraulic problems not often encountered. Nevertheless, these problems have been studied, as previously stated, for more than thirty years, by some of the ablest engineers in this country and the basic principles and main features have been agreed upon. In the reports mentioned, it is stated that 0.1-ft. elevation on the lakes was worth $660 000 per year to the navigation interests. This statement has never been contradicted and must be assumed correct until disproved. Moreover, there seems to be little doubt of the ability to provide at least a foot more water in Lake Erie (with the corresponding increase in elevation of Lakes Huron and Michigan) by the construction of remedial works. The speaker cannot understand, why, with a probable value of more than $6 000 000 per year in sight, there should be any question about the advisability of constructing the works required. What is now needed is that these facts be brought to the attention of those in authority and that those interested demand construction at once.

---

[7] Cons. Engr., Buffalo, N. Y.

E. W. LANE,[8] M. AM. SOC. C. E. (by letter).—The control of the lake levels is a most interesting problem. The writer worked on the study of the regulation of the Great Lake levels made by the Joint Board of Engineers for the St. Lawrence Waterway, which was under the immediate direction of Colonel Pillsbury. He entered upon these studies with a predilection for the improvement of the existing condition by the construction of regulation works for the benefit of navigation and power development. This seemed better than the compensation of the lake levels for diversions by placing fixed obstructions in the connecting streams. Such fixed obstructions would benefit navigation by increasing the navigable depths, but would be of no advantage in the development of water power.

As a result of the studies, the writer is convinced that the Joint Board of Engineers got to the bottom of the matter, and shares with the author the view that the benefits at the present time (1929) do not justify the cost of regulation works. Since this opinion was stated by the Board, the Supreme Court decision on the Chicago diversion, and the rapid rise of lake levels, have further strengthened the conclusion.

The studies made heretofore for the regulation of the Great Lakes have been with the view of improving the lake levels for navigation, or of causing an increased flow in the Niagara and St. Lawrence Rivers in low-water periods, or to accomplish both these results. Another benefit of regulation is possible, however, but so far as known, no studies have been made of the practicability of regulation for this end.

Within transmission distance of the St. Lawrence and Niagara water powers are many existing and possible hydro-electric developments on streams which have a large flow at certain seasons of the year and low flow at others. The outflow of the Great Lakes varies in much the same manner as these streams, but it might be feasible to use the lakes, particularly Erie and Ontario, to store water during the season when the other streams had an excess, and discharge from storage when they were low. By the interchange of power an equalization of supply might be secured which would be of sufficient value to justify the expense of control works. In fact, it seems probable that some such result may come about as a natural sequence of events. When the International Section of the river is developed, it will probably be by means of a single dam at the lower end. For some years after this is completed there will be a surplus of power and a market for at least part of it will be found in supplying existing hydro-electric systems with power during their low-flow periods.

---

[8] Research Engr., U. S. Bureau of Reclamation, Denver, Colo.

As the demand for power approaches the capacity of the International Section an effort will be made to continue to supply power during the low-flow season, to the existing hydro-electric systems, for since the value of secondary power is much less than that of primary power, the value per kilowatt-hour of power which will combine with the secondary power and provide a continuous supply, is considerably greater than the value of primary power. With this condition of the power demand there will be an immediate return on an investment in regulation works; and because of the immense quantities involved, they may be economically justified, especially since Lake Ontario will be already controlled by the power plant in the International Section and control works for Lake Erie are comparatively inexpensive. Because of the rapid changes taking place in the electrical power field, one would be rash to predict anything so far ahead as this condition must necessarily be; but to state that regulation would never be feasible would be equally rash.

F. G. Ray,[9] M. Am. Soc. C. E. (by letter).—The regulation of the Great Lakes has been a subject of discussion for many years. At first glance it appears to be a simple engineering proposition. There are, however, opposing interests to be served and restricting conditions to be met which offer many complications. The author has clearly set forth the fundamental considerations in reaching a solution and his conclusions show what little betterment may be expected in attempting to improve upon the regulation that Nature has provided. Regulation is feasible as he has shown, but it must be kept within a narrow range to avoid unreasonable damage to those whose interests are diametrically opposed to the interests benefited. The net result is small as measured by the cost. The time may come when regulation is warranted, but in the writer's opinion that time has not yet arrived.

The subject is highly interesting and should commend itself to the study of hydraulic engineers. If regulation is ever to be adopted, every practicable plan should be conceived and considered in advance. Regulation of Lake Superior is now in successful operation. The program to be followed and the benefits to be derived with regulation of Lakes Erie and Ontario are matters for study, but the design and construction of regulating works for the Niagara and St. Lawrence Rivers offer no serious difficulties. Regulation of Lakes Michigan and Huron, however, is a more intricate problem. These lakes discharge through 90 miles of channel which carries a flow of 200 000 cu. ft. per sec., with an average slope of less than 0.1 ft. per mile. There is an immense traffic through this channel, which must not be seriously hampered by obstructions or largely increased velocities. These are conditions that must be met in regulating the two lakes in question.

In analyzing the practicability of regulating schemes it has been customary to test them by use of supply factors and lake levels of record since 1860. The plans discussed by Colonel Pillsbury were so tested by him up to 1925. The writer has extended the so-called partial regulation scheme for Lakes

---

[9] Prin. Engr., U. S. Lake Survey, Detroit, Mich.

Michigan, Huron, and Erie through to the end of 1929. It is interesting to note that the high-water peak which resulted in considerable damage on Lakes Michigan and Huron in 1929 would have been exceeded by 1 ft. and Lake Erie would have gone 3 in. above the highest previous level since 1860, if this particular plan had been in operation. Regulation projects based on supply factors now available are accordingly further restricted or subject to less favorable results.

The author has shown the effect of Lake Superior regulation on the levels of the several lakes from 1914 to 1925, inclusive. This study has also been extended to include 1929. It has been found that the maximum stage on Lake Superior in 1928 was about 7 in. below what the unregulated level would have been; also that the effect of the regulation on the maximum stages of Lakes Michigan, Huron, and Erie in 1929 was to raise each of these lakes slightly less than 2 in.

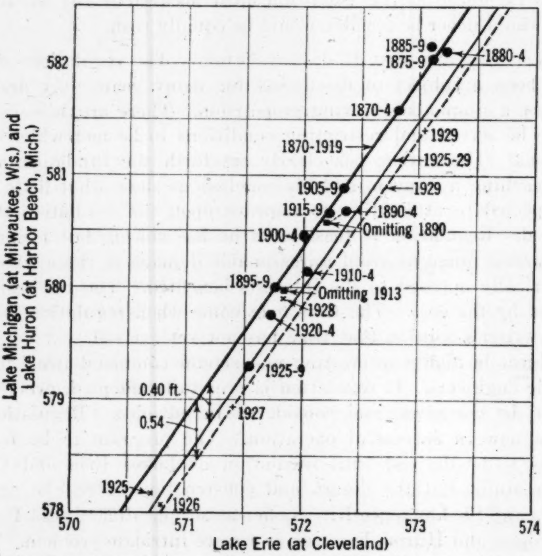


FIG. 14.—COMPARISON OF LEVELS IN LAKES MICHIGAN-HURON AND LAKE ERIE.

In proposals for compensating works in the outlets of the several lakes, it is usually considered that such works should be designed so as to compensate for all losses in lake level due to diversions and to changes in original capacity of outlet. The amount that Lake Erie has been lowered and the corresponding amount of compensation required in the Niagara River are known with little uncertainty. In the case of Lakes Michigan and Huron the effect of diversions are determined by means of measured increments of flow in the outlet. The possibility of changes in capacity of outlet and the lowering effect on Lake Huron of channel enlargements in the St. Clair and Detroit Rivers, prior to the beginning of discharge measurements in 1899, cannot be determined by direct observation, except through comparison of past and present relation of levels on Lake Huron and Lake Erie. The period of relatively low water from 1890 to 1926 left considerable doubt as to the showing of such comparison. The high stages of 1929 have done much to strengthen the evidence.

Fig. 14 shows a comparison of lake levels by plotting 5-year means with the levels of Lakes Michigan and Huron as ordinates and the levels of Lake Erie as abscissas. The observed levels have been corrected for earth tilt to corresponding levels at the outlets. It will be noted that 5-year means from 1870 to 1919 show little departure from a straight-line relationship, except for the two groups, 1890-1894 and 1910-1914. The local supply to Lake Erie was notably excessive in 1913. Lake Erie levels in 1890 appear relatively high. The reason has not been investigated. When the two years, 1890 and 1913, are omitted, the levels for the remaining 48 years of the 50-year period are remarkably consistent, and show no evidence of change in relative height, the levels for 1925-1929, however, fall about 5 in. below the 1870-1919 line and the single years, 1926, 1927, and 1929, are about 6½ in. below this line. The 1929 levels are well up among those of early years. The evidence indicates that Lake Huron held its proper level until after 1919 and has been lowered since that time by about 6 in. This amount is subject, of course, to some error resulting from inequalities in local supply.

Soundings in the head of St. Clair River in 1925 showed a marked enlargement in cross-section due to gravel dredging in the vicinity of Point Edward and below. The amount of enlargement indicates a resultant lowering of about 4 in. on Lake Huron. Discharge measurements and analysis of slopes show the effect to be about 3½ in. The amount of lowering not accounted for by this enlargement is apparently not more than about 3 in. A continuation of high water on the lakes will afford safer and surer conclusions. However, in the writer's opinion, the effect on Lake Huron levels of channel improvement and other changes in the St. Clair and Detroit Rivers is now known quite definitely.

George B. Pillsbury, M. Am. Soc. C. E. (by letter).—This paper was prepared to introduce and invite discussion on lake regulation as affected by the Niagara River. In addition to the discussions presented, attention is invited the able treatment of the regulation of Lake Erie with reference to power, contained in the paper by Norman R. Gibson, M. Am. Soc. C. E., entitled "Niagara Power"[10] and in discussion[11] thereon by William Kelly, M. Am. Soc. C. E.

To the Engineering Profession at large, as well as to the general public, the feasibility and desirability of the regulation of the lakes appear axiomatic. The writer must confess that he approached the subject, some years ago, in a more than hopeful frame of mind. It is difficult to appreciate the extent to which widespread interests have adjusted themselves to the present regimen of the lakes and their outlets, and the extent to which injury, more or less weighty, would be created by even small changes in this regimen.

---

[10] See pp. 16 *et seq.*

[11] See p. 41.

A feature of lake regulation also difficult of appreciation, which was brought out by Mr. Gibson,[12] is that a system of regulation by which the discharge increases uniformly with the elevation of the lake will afford the least volume of high discharge and the greatest volume of useful flow. This is the system of regulation afforded by the natural outlets. An examination of the results secured by Mr. Horton's suggested regulation of Lakes Michigan and Huron, in Fig. 13, shows that while the extreme low flow is increased, the ordinary flow is decreased for a much longer part of the time. The duration curve of Mr. Horton's regulated outflow would unquestionably take the same general form as that indicated for the regulated flows of Lakes Erie and Ontario in Fig. 3.

A third feature of regulation of the lakes, which is generally overlooked, is the limited discharge capacity of the outlets. In Mr. Horton's plan, it is proposed, for example, to discharge 250 000 cu. ft. per sec. from Lake Huron when the elevation of that lake is 580.2 ft. above sea level. Assuming that Lake Erie is even 9 ft. below Lake Huron, the St. Clair River can not discharge as much as 200 000 cu. ft. per sec. when Lake Huron is at this level. The enlargement necessary to increase the discharge capacity of the St. Clair River by 25% might well cost more than the deepening of all the channels and harbors of the lakes by an amount equivalent to the maximum benefits from any suggested system of regulation. It is to be recollected that the outlets to the lakes are now, and steadily have been, discharging water just as fast as they possibly can, but that at present the lakes are nevertheless higher than some would like. The cost of enlarging the outlets—all the way down to Montreal—to a degree sufficient to afford a comfortable margin for regulation, is one of the reefs on which proposals for regulation are shipwrecked.

Mr. Horton explains very clearly the theoretical objections to controlling works as a remedy to the causes of impaired lake levels. It is to be observed, however, that in Fig. 10, the lines, AI and DE, when considered in their entirety as extending to the virtual sills of the outlet, are not right lines but are concave downward. In any practical application of compensating works, to correct the relatively small causes that have resulted in lowering the lakes, the two curves are so nearly parallel, in the region comprised between the range of lake levels, that the distances, KL and GH, are inappreciable.

Especial attention may be invited to Mr. Ray's discussion, with which the author completely concurs. It is hoped that the interesting discussions that have been presented will afford further light on this moot engineering question.

---

[12] See p. 18.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1765

EXPERIMENTAL RESEARCH ON VIBRATION DAMPERS AND INSULATORS[1]

By Stephen E. Slocum,[2] M. Am. Soc. C. E.

With Discussion by Messrs. J. P. Den Hartog, Francis G. Tatnall, W. M. Dawley, and Stephen E. Slocum

Synopsis

The study of vibration insulation presented in what follows is an original research undertaken in connection with the general problem of preventing vibration in structures, as applied in particular to the terminal improvements of the Pennsylvania Railroad at Philadelphia, Pa., now (1928) under construction.

In order to study the transmission of vibration it was necessary to have two vibrating systems to act as transmitter and receiver, respectively. Each of these elastic systems had its own vibration characteristics. Moreover, the insulator placed between them was also elastic and resilient and, therefore, added a third vibrating element. Loads were also transmitted through this threefold composite system, which had a further effect in modifying the vibration characteristics.

It is a notable fact, however, that no practical solution has yet been found of the vibration characteristics of such a primary element as a simple beam of uniform cross-section under load. Simplifying assumptions, such as neglecting the mass of the beam, or assuming that it is unloaded, have always been found necessary to a mathematical solution, and these make the results of such analysis of little practical value.

Moreover, such experimental studies of vibration insulators as have appeared, have also been of a very special nature and so limited in scope as to warrant no general conclusions. It is evident, therefore, that the present

---

[1] Published in October, 1929, *Proceedings*.

[2] Cons. Engr., Ardmore, Pa.

study is of a somewhat pioneer nature in a difficult field of growing practical importance. It is believed that the essential features of the problem have been kept distinct, and that practical results have been brought to light concerning the general problem of vibration control.

VIBRATION CONTROL

The use of insulation for creating a short circuit in vibration in structures is only one phase of the complicated problem of vibration control. The general problem, whether considered in relation to structures or machinery, is a dynamic rather than a static one. Probably it is for this reason that greater progress toward a solution has been made in mechanical than in structural lines.

The general problem of vibration control may be classified under several heads. In the design of permanent machinery installations, such as lighting and power units in office buildings and apartments; batteries of looms, printing presses, or machine tools in shops and factories; marine installations; automotive and aircraft power plants, etc., the first step toward a solution consists in eliminating the unbalanced effects in rotating parts by means of dynamic balancing.

This alone, however, is in general only partly effective. In reciprocating machinery where the power fluctuates, it is of the utmost consequence to design parts affected by vibration so as to avoid critical speeds. In relation to building construction, critical speeds are those at which synchronous vibrations are set up in one or more members of the structure. When vibration is due to permanent installations, such synchronous vibrations may be avoided, as in machinery, by properly designing individual structural members, such as columns and girders.

The third general principle, applying particularly to structural work, consists in causing the vibration to become "short-circuited" by confining it to the region where it originates. For example, a machine may be mounted on a structure entirely independent of those adjacent to it; various types of insulation may be interposed between the structures; nodes may be artificially created at points of contact; or use may be made of existing nodes in the structure, the latter in particular being a very useful method in marine installations.

Mechanical insulators fall into two general classes: Those that are elastic and resilient, such as cork, felt, and rubber; and those that are almost or entirely lacking in these qualities, such as asbestos, lead, sand, and gravel. Properly speaking, there is no such thing as a vibration absorbent, since this implies dissipation of energy through internal work in the form of molecular friction, heat losses, etc. Resilient materials store up energy like a spring, and give out this energy again when released without appreciable loss; and the damping properties of non-resilient materials, such as sand and gravel, depend on destroying the rigidity, or interrupting the continuity, of otherwise rigid structures.

DESCRIPTION OF APPARATUS

For the purpose of this study it was essential to have two elastic systems, one of which could be set in vibration mechanically and would serve as transmitter, while the other could be insulated from the first by the material under test, and would serve as receiver of such vibration as passed through the insulation. It was also considered essential to provide for a considerable range of loads on the test piece, and for varying the frequency of the forced vibration within practical limits. The general arrangement of the test apparatus designed to meet these requirements is shown in Fig. 1.

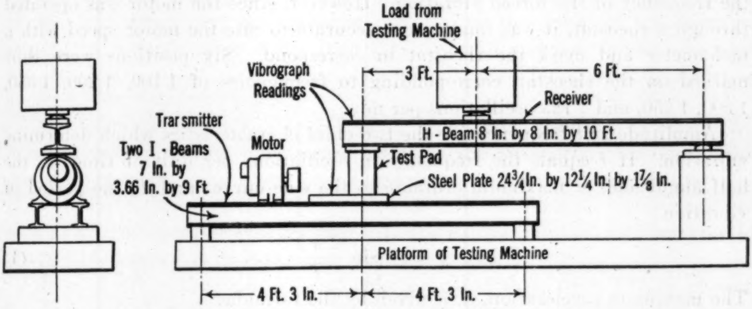

Load from Testing Machine
Vibrograph Readings
3 Ft.
6 Ft.
Receiver
Transmitter
Two I-Beams 7 In. by 3.66 In. by 9 Ft.
Motor
H-Beam 8 In. by 8 In. by 10 Ft.
Test Pad
Steel Plate 24¾ In. by 12¼ In. by 1⅞ In.
Platform of Testing Machine
4 Ft. 3 In.
4 Ft. 3 In.

FIG. 1.—ARRANGEMENT OF TESTING APPARATUS.

To secure the desired loading, and also a long platform on which to set up the test apparatus, a large testing machine, with a platform 20 ft. long, was chosen as a base for the experiments. The primary vibrating system, or transmitter, consisted of two 7-in., 15.3-lb., I-beams, 9 ft. long, mounted parallel to each other, and lengthwise of the platform of the testing machine. On this pair of I-beams, a 1-h.p. electric motor, with an unbalanced driving pulley, was clamped, to produce the forced vibration. At the center of the beams and adjacent to the motor a machined steel plate was clamped lengthwise to form a platform for the insulating material under test (Fig. 1). On this platform, beside the test pad, was placed the "vibrograph" by which the amplitude of the forced vibration was recorded.

The unbalanced effect on the driving pulley of the motor was obtained by removing one of the bolts by which the pulley was attached to the flange and replacing it with a longer one carrying a heavy nut at each end. The motor speed, or frequency of the forced vibration, was controlled by a rheostat giving a speed range from 1 100 to 1 725 rev. per min.

Directly over the transmitter, an 8-in. 31-lb., steel **H**-beam, 10 ft. long, was mounted so that one end was supported on the insulating material under test, and the other end blocked up on the platform of the testing machine. This **H**-beam constituted the secondary vibrating system, or receiver.

The load from the testing machine was applied to this **H**-beam through a spherical bearing-block. By this means both the static load and the dynamic vibration were transmitted in all cases through the insulating material under test.

VIBROGRAPH RECORDS

The graph traced by the vibrograph was a simple sine or cosine curve, without harmonics, which represents the simplest possible type of vibration. Throughout the entire series of tests the ratio of amplification was 12; that is, the amplitude of the graph traced by the instrument was twelve times the amplitude of the actual vibration. As reproduced in this paper, the vibrographs are six times the actual scale of amplitude, or one-half the full size as traced.

The instrument has an auxiliary pen that traces a graph of known frequency, and this is used ordinarily as a base for determining, by comparison, the frequency of the forced vibration. However, since the motor was operated through a rheostat, it was found more accurate to rate the motor speed with a tachometer and mark the rheostat to correspond. Six positions were thus marked on the rheostat, corresponding to frequencies of 1 100, 1 200, 1 350, 1 500, 1 650, and 1 725 oscillations per min.

Amplitude and frequency are the two chief characteristics which determine vibration. If f equals the frequency in oscillations per unit of time; a, the half amplitude, or maximum ordinate to the sine curve; and T, the period of vibration:

$$y = a \sin \frac{2 \pi t}{T} \quad \text{(1)}$$

The maximum acceleration, α, is given by the formula:

$$\alpha = 4 \pi^2 f^2 a \quad \text{(2)}$$

Since acceleration is the fundamental quantity of dynamics, the vibrograph records evidently determine fully the characteristics of the vibration.

The readings were taken by placing the instrument alternately on the test platform of the transmitter beside the test pad, and on the upper flange of the H-beam, or receiver, directly over its point of support on the test pad. In each position a graph was recorded at each of the six speeds to which the rheostat was calibrated. Each test at a given load, therefore, comprised two sets of six vibrograph records, or twelve graphs altogether.

Moreover, the tests covered a considerable range of loads. For materials like sand, gravel, lead, and asbestos, the test loads as measured on the scale-beam of the testing machine ranged from 300 to 24 000 lb., corresponding to a maximum unit intensity of loading on the test pad of 450 lb. per sq. in. In some cases this schedule required a total of ninety-six vibrograph records for one complete test. For corkboard and similar materials on which the load range was more limited, a complete test usually totaled forty-eight records.

The speed of the drum carrying the paper roll on which the graph was traced, was regulated by a governor which maintained the angular velocity constant. The linear speed of the paper passing under the pen, therefore, depended on the diameter of the roll. The graph thus appears more extended in some cases than in others, depending on the diameter of the paper roll when the record was taken. This does not affect the frequency, since the frequency was determined by calibrating the rheostat.

INTERPRETATION OF GRAPHS

The test apparatus was assembled from commercial units, subject to the usual variations in uniformity of shape and loading actually met in practice.

The results are, therefore, more typical of actual conditions in engineering than would have been the case with apparatus of greater refinement, such as might be available in a physics laboratory.

If a single load or a single frequency had been adopted, doubtless the results would have shown a greater uniformity, but it is precisely this lack of uniformity that is met in practice. A train entering or leaving a terminal or subway station is subject to a continual variation in speed which often causes a severe vibration to appear at one instant and disappear the next. A live load passing over a structure also causes the load in any given member to vary within relatively wide limits. Such varying effects, due to continual change in load and frequency, are therefore essential to the study of vibration in structures. Graphs taken under such varying conditions also serve to disclose the complexity of the vibration problem and the need for systematic research and specialization in this field.

The so-called impact load in bridges is a dynamic effect due to vibration; it arises not only from unbalanced forces in the locomotive, but also from periodic forces set up by wheel loads succeeding each other at regular intervals. It is apparent therefore that the relation of load to vibration characteristics, such as critical speed or synchronism with the natural frequency of structures, is of great importance in many branches of engineering.

About 2 000 records were taken in the course of these tests. Those chosen for illustration in this paper are typical, and display certain characteristic properties of the material or loading.

It is evident on inspection that load has a marked effect in modifying vibration characteristics. Thus, Fig. 2(*a*) and Fig. 2(*b*), taken with the same unit load, exhibit similar characteristics; whereas Fig. 3(*a*) and Fig. 3(*b*), taken with the same insulating pads, but with a different unit load, show quite different characteristics. In Fig. 3, the motor speed is synchronized with the natural frequency of the transmitter.

Frequency has little effect on the damping properties of insulation—that is, on the percentage of amplitude damped out by the pad—except at critical points. The percentage of damping, however, depends very largely on the kind of insulation, or the material of which the pad is made, as will appear more fully later.

In interpreting the vibrographs, therefore, the effect of load, material, and frequency must be clearly distinguished in making comparisons.

CORKBOARD INSULATION

The insulating properties of any type of resilient pad is a function of its deflection under load. Experience has shown that in order to secure the best results in machinery insulation it is important to adjust the dimensions of the pad to the load and, also, to the speed of the machine, or frequency of vibration. Either the thickness of the pad may be changed, keeping the area constant; or the area may be altered, keeping the thickness constant. In either case, the deflection of the pad under load is affected, which is the chief quality on which its properties as a vibration damper depends.

Corkboard is widely used for vibration insulation, and is commercially standardized for this purpose as to density and dimensions. For test purposes blocks of corkboard were used, 6 by 6 in. in area by 3 in. thick. They were of standard density equal to 0.319, which is equivalent to a unit weight of 1.66 lb. per board-ft. The compressibility of this material under load is given by the load-deflection curve shown in Fig. 4, plotted from test data obtained from one of these pads.

The corkboard blocks were piled up so as to give four thicknesses for vibration tests, namely, 3 in., 6 in., 9 in., and 12 in. Each pad was tested for a series of loads, and vibrographs were taken at six frequencies for each load. The results obtained with two of these thicknesses of corkboard pad under a unit load of 27 lb. per sq. in., are shown in Fig. 2.

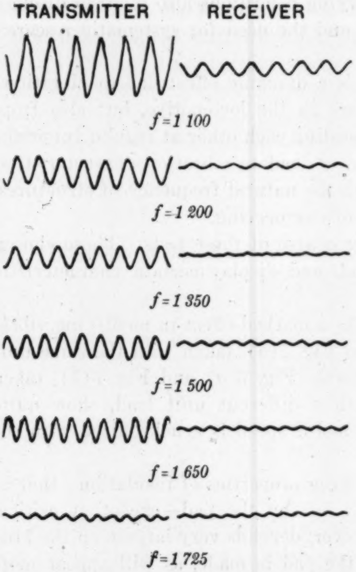


FIG. 2(*a*).—VIBROGRAPH OF CORKBOARD, 6 BY 6 BY 6 IN. (UNIT LOAD, 27 LB. PER SQ. IN.)

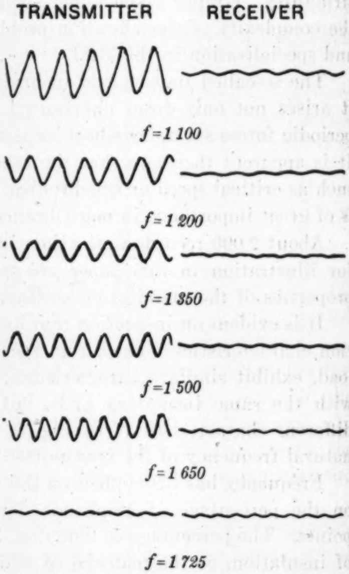


FIG. 2(*b*).—VIBROGRAPH OF CORKBOARD, 6 BY 6 BY 12 IN. (UNIT LOAD, 27 LB. PER SQ. IN.)

If T denotes the amplitude of vibration of the transmitter, and R, the amplitude of vibration of the receiver, then $k = \dfrac{R}{T}$ may be assumed to indicate the amount of damping. With this notation, the percentage of amplitude of vibration damped out by the pad becomes $D = 100\ (1 - k)$. By scaling the amplitudes from the vibrographs and determining the damping percentage it is found that, for the first five frequencies, the damping percentage for a given depth of pad is practically constant. The average values of D for these five frequencies are plotted in Fig. 5.

It is further evident from Fig. 2 that, at the highest frequency, namely, 1 725 oscillations per min., the amplitude of vibration of the transmitter

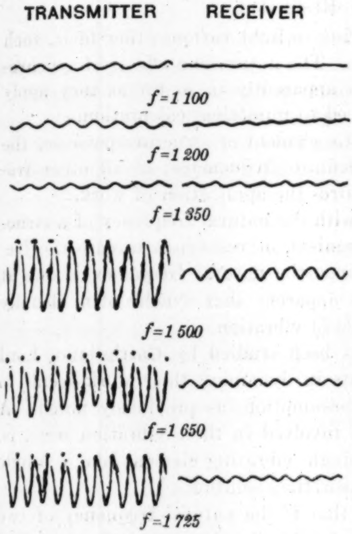


FIG. 3(*a*).—VIBROGRAPH OF CORKBOARD, 6 BY 6 BY 6 IN.

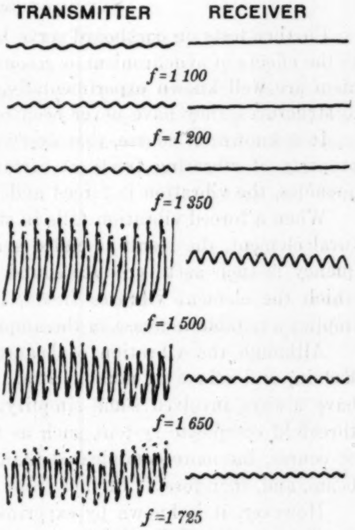


FIG. 3(*b*).—VIBROGRAPH OF CORKBOARD, 6 BY 6 BY 12 IN.

decreases greatly and suddenly, while that of the receiver is not materially affected. This does not mean that the damping effect is lost, but that this particular frequency corresponds to a critical point, or node, of the transmitter. This further indicates the value of carrying such vibration experiments over a considerable range of loads and frequencies, because the results obtained at a critical point have a special rather than a general significance.

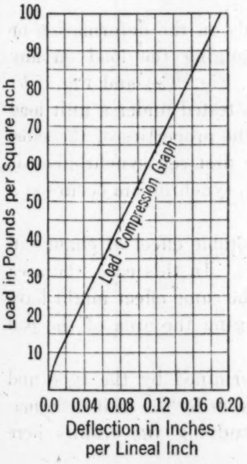


FIG. 4.—COMPRESSIBILITY OF CORKBOARD.

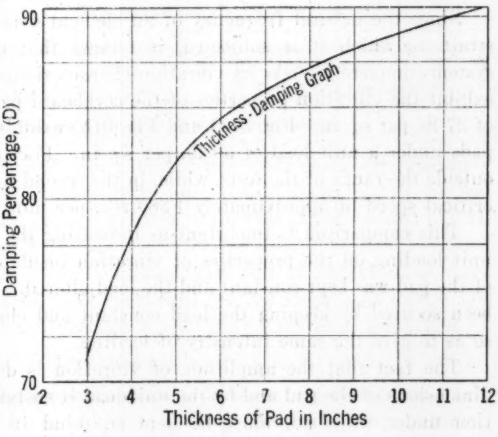


FIG. 5.—EFFECT OF THICKNESS ON DAMPING.

SYNCHRONISM OR RESONANCE

Further tests on corkboard serve to bring to light various other facts, such as the effects of synchronism or resonance. The nature and effects of synchronism are well known experimentally, but apparently in so far as they apply to structures, they have never been reduced to numerical calculation.

It is known, of course, that every elastic element or structure possesses the property of vibrating freely at certain definite frequencies; at all other frequencies, the vibration is forced and requires the application of work.

When a forced vibration falls in step with the natural frequency of a structural element, the result is called synchronism, or resonance in case the frequency is such as to produce sound. Since the natural frequency is one at which the element vibrates freely, it is apparent that synchronism usually implies a notable increase in the amplitude of vibration.

Although the vibration of beams has been studied by Castigliano, Lord Kelvin, and others of notable attainments in dynamics, their work seems to have always involved some simplifying assumption, as previously noted. A threefold composite system, such as that involved in these vibration tests, is, of course, far more complicated than a single vibrating element like a simple beam, and, therefore, is further from a theoretical solution.

However, it is known by experiment that if the natural frequency of two elastic systems are denoted by f_1 and f_2, respectively, the natural frequency of the composite system, in general, will be neither f_1 nor f_2. An insulating pad, such as those tested, also has a marked effect on the natural frequency of the composite system. This effect of insulation was studied by Berger,[3] who found that the nature of the material, as well as the area and thickness of the pad, affected the amplitude of vibration and the natural frequency, or point of synchronism. He found, for example, that at a given frequency, an insulating pad of improper dimensions might serve to intensify the amplitude of vibration.

Since the natural frequency of an element depends on the deformation or strain to which it is subjected, it follows that changing the load on any system completely alters its vibration characteristics. Fig. 2(*a*) and Fig. 2(*b*) exhibit the vibration properties of the corkboard pads tested under a unit load of 27 lb. per sq. in. Fig. 3(*a*) and Fig. 3(*b*) exhibit the properties of the same pads under a unit load of 50 lb. per sq. in. For the first set, synchronism is outside the range of the tests, while, in the second test, synchronism occurs at a critical speed of approximately 1 500 rev. per min.

This comparison is important as indicating the notable effect of change in unit loading on the properties of vibration insulators. In this case, the area of the pad was kept constant and the load altered. The same effect might have been secured by keeping the load constant and changing the area of the pad so as to give the same intensity of loading.

The fact that the amplitude of vibration is determined by the type and dimensions of the pad and by the unit load it sustains, or by the unit deformation under the load, must be kept in mind in studying the results here presented.

---

[3] *Gesundheits Ingenieur,* 1913, Vol. 36, p. 433.

HAIR FELT INSULATION

Hair felt is very commonly used for sound absorption, and, to a more limited extent, as vibration insulation for machinery.

The basis of the felt used in these tests is said by the makers to be animal hair, compressed and milled by special processes, and sealed by a proofing solution which enables it to withstand high unit compression; it also increases its resiliency, and makes it damp-proof. When used as machinery insulation, the surface is further protected by applying a special oil, water, and fire-resisting paint.

The samples were 3 by 2 in. in area, and in four thicknesses, namely 1 in., ½ in., ¼ in., and ⅛ in. The first two samples were used together, making a test pad, 3 by 2 by 1½ in. in size. The average density was 0.397, which is equivalent to a unit weight of 2.066 lb. per board-ft. Its properties under load are apparent from the load-deflection curve shown in Fig. 6(*a*).

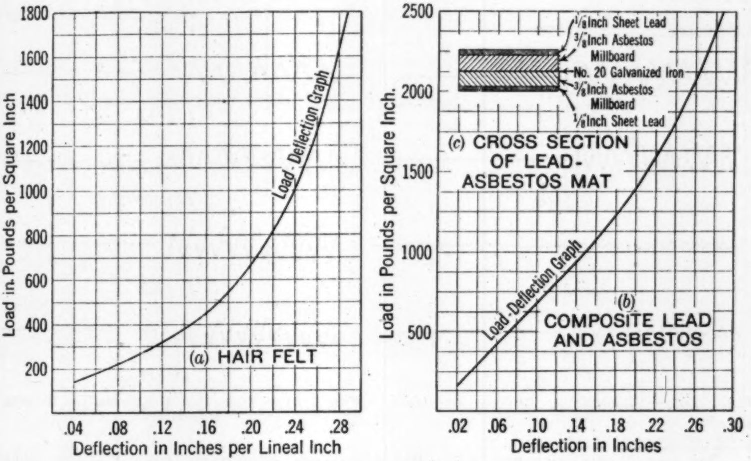


FIG. 6.—PROPERTIES OF TWO TYPES OF VIBRATION MATS.

As these samples were so small, the range of test loads was correspondingly limited. For comparison with corkboard, a unit load of 50 lb. per sq. in. was applied (see Fig. 7(*a*)). The average value of k for this set of records is 0.21, and the damping percentage is 79.

This result indicates that well-fabricated hair felt is considerably more effective as a vibration damper than corkboard for the same unit load. The material will also withstand considerably higher unit loads than corkboard.

LIVE RUBBER

This material has resilient properties similar to those of corkboard and felt. Tests were made on a special type of rubber bushing used for engine mounting, both for the practical interest of such use of rubber, and also to test the claim that it possesses better damping properties in shear than in compression.

As shown in Fig. 8(*b*), these mountings comprised inner and outer metal sleeves, united by a moulded rubber bushing, through which the load was transmitted in shear from one steel sleeve to the other. These bushings are intended to be mounted axially, putting the rubber in shear and loading it symmetrically and uniformly. Evidently, they would be much less effective as vibration dampers if mounted radially, since only a small part of the rubber would be in action, and even on this segment the load would not be distributed uniformly.

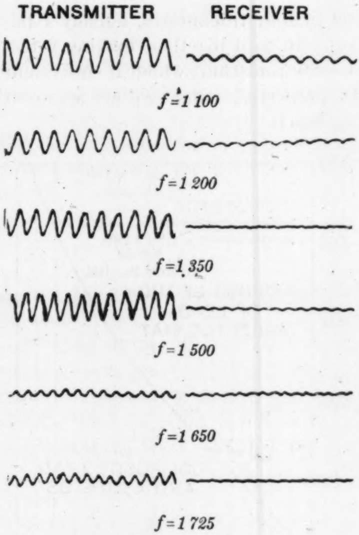


FIG. 7(*a*).—VIBROGRAPH RECORD OF HAIR FELT.

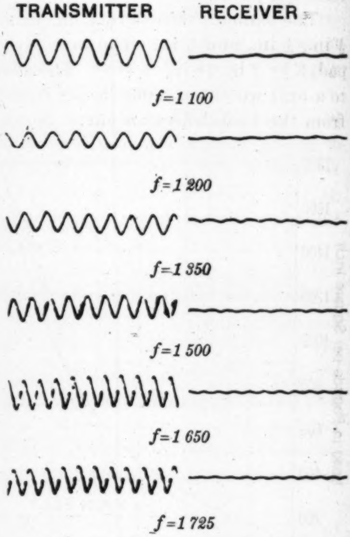


FIG. 7(*b*).—VIBROGRAPH RECORD OF RUBBER BUSHINGS.

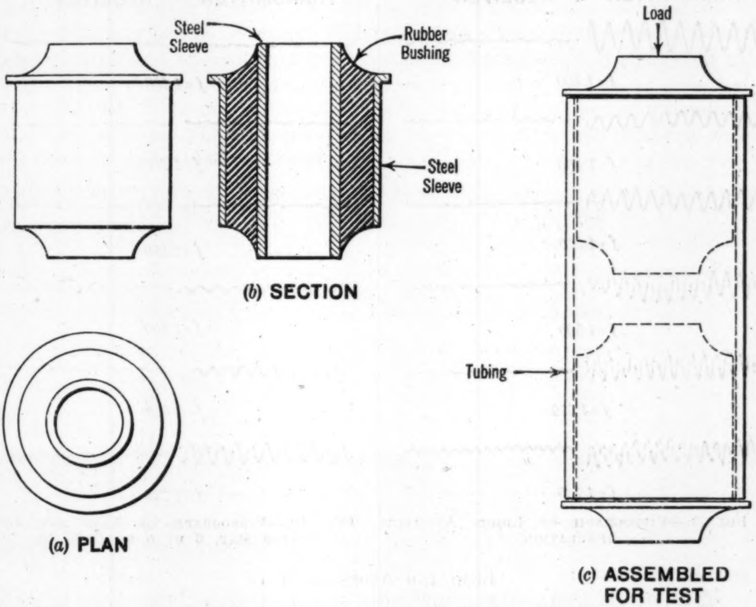
To test these bushings with the apparatus shown in Fig. 1, they were mounted in pairs in a short section of pipe, as shown in Fig. 8(*c*). Two such pairs, or four bushings, were then used as legs for supporting the end of the H-beam on the test platform. With this arrangement, each pair carried one-half the reaction at the end of the H-beam, and this load passed through two bushings. Each one thus carried exactly the same load, but as the vibration passed through two bushings in series, the amplitude of vibration was damped twice as much as for a single bushing.

The manufacturer's rated loading for this type of bushing is 150 to 200 lb. per bushing. They were tested as vibration dampers in the manner just explained under three different loads, the vibrograph records in Fig. 7(*b*) being for a load of 224 lb. per bushing.

The average damping factor determined from this test was 0.096 per pair in series. For a single bushing, therefore, the value would be approximately twice this amount, or $k = 0.19$. Consequently, the percentage of damping for bushings of this particular type and loading is found to be 81. For a load of 160 lb. per bushing, the damping percentage was found to be 85.

ASPHALT EMULSION

One of the most important forms of vibration insulation in structural work is lateral insulation around piers and column footings. For this purpose a light grade of corkboard is often used because the insulation is introduced simply as a separator and transmits no load. Plastic or semi-fluid materials might also be used as lateral insulation, simply poured into cracks. With this in mind, an asphalt emulsion was tested to determine its properties as a vibration damper. This was received in a semi-fluid condition.



(a) PLAN (b) SECTION (c) ASSEMBLED FOR TEST

FIG. 8.—RUBBER BUSHING USED FOR VIBRATION TESTS.

To test the material in this state, it was necessary to confine it in a rectangular tin can, 9 by 6½ by 4 in. This container was filled with the asphalt emulsion until it bulged and was then placed on the test platform with its 4-in. dimension vertical. A square steel plate, 6 by 6 by 1½ in., was placed on the can, and this, in turn, supported the end of the **H**-beam, or receiver. In shape and dimensions, this test pad was, therefore, very similar to the corkboard pads tested. In the case of lateral insulation, the load is nominal. A light load of 396 lb. (11 lb. per sq. in.) was applied therefore to the steel plate. The vibrograph records for this test are shown in Fig. 9. These give an average value of $k = 0.113$, or a damping percentage of 88.7.

This material dries out in air, however, and although it does not completely harden, but remains plastic or sticky, it possesses little or no resilience. Therefore, although it would not transmit vibration like a rigid body under any conditions, its effectiveness as a vibration damper would be relatively small.

However, this test was otherwise useful as confirming the results obtained with resilient materials, such as corkboard, felt, and rubber, since the can with its semi-fluid contents was highly resilient, offering very slight resistance to deformation and resuming its shape instantly on removal of load. The results of this test, therefore, confirm fully the general conclusion that the qualities essential to an effective vibration insulator are slight resistance to deformation and complete resilience.

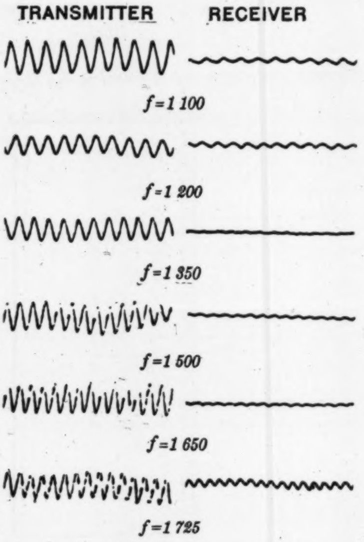


Fig. 9.—Vibrograph of Liquid Asphalt Insulation.

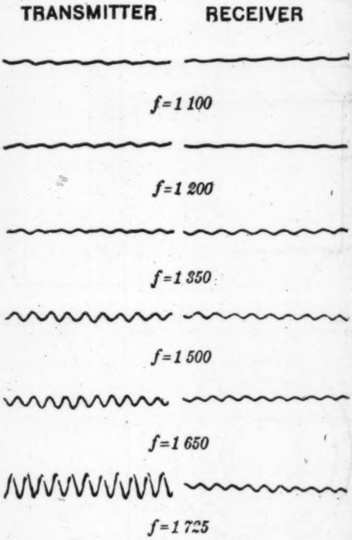


Fig. 10.—Vibrograph of Lead and Asbestos Mat, 6 by 6 by 10.75 In.

Lead and Asbestos Mats

Composite pads formed of alternate layers of lead and asbestos have been used extensively by the New York Central Railroad Company in the Terminal Zone, New York, N. Y., as vibration mats under column footings. A large number of tests, therefore, were made on such pads to determine their properties as vibration dampers, as well as their physical properties with respect to resilience and deflection or lateral deformation under load. The pads tested were identical in make-up with the vibration mats used in the construction of the Graybar Building,[4] New York, which were originally devised as sound deadeners. They have proved effective in eliminating low pitched noises of approximately 50 oscillations per sec.

---

[4] *Engineering News-Record,* February 10, 1927, pp. 228–230.

The standardized mat adopted in this work consists of five separate layers, as shown in Fig. 6(*c*). The bottom layer of sheet lead is turned up and folded over the top layer as a protection against water. Care was taken, in making up the test pads, to use the same quality of asbestos millboard, from the same manufacturers, as specified for the vibration mats used in the Graybar Building. The total thickness of such a composite pad is approximately $1\frac{1}{16}$ in.

To determine this deformation under heavy loads, a sample pad, 6 by 6 in. in area, was tested in compression up to a total load of 100 000 lb., which is equivalent to a unit load of 2 778 lb. per sq. in. The load-deflection curve obtained from this test is shown in Fig. 6(*b*). No lateral spreading of either lead or asbestos layers could be detected at any load.

A second test was made on a similar pad to observe the effect of time on deflection. The unit intensity of loading on the pads used in the New York Central Zone is said to be about 500 lb. per sq. in. To cover this and a somewhat higher range of loading, a total load of 30 000 lb., which is equivalent to a unit load of 833 lb. per sq. in., was kept on a test pad, 6 by 6 in. in area, for 75 hours. The thickness of the pad at the beginning of the test under nominal load was 1.088 in. The scale-beam of the testing machine was balanced, and the thickness of the pad measured three times daily. The results of this test were as follows:

Elapsed time, in hours.	Thickness, in inches.
0	1.022
3	0.997
7	0.994
23	0.993
27	0.991
31	0.991
47	0.990
51	0.989
55	0.989
71	0.988
75	0.988

On removal of the load the pad returned immediately to a thickness of 1.039 in.; after 24 hours, it was 1.060 in.; after 48 hours, it was 1.076 in.; and after 96 hours, it was 1.086 in.

These pads were so extremely solid that, with the test apparatus used, no damping effect could be observed with a single pad, 6 by 6 by 1 in. With four such pads piled together, making a test block, 6 by 6 by $4\frac{3}{16}$ in., the damping effect was still so slight that its numerical value could not be definitely determined.

To amplify the damping effect sufficiently to get more pronounced and reliable results, ten such pads were piled together. In two of these pads the thickness of the asbestos layers was $\frac{1}{2}$ in. instead of $\frac{3}{8}$ in.; in all other respects, the pads were exactly identical. This composite tenfold test pad, comprising altogether 40 layers of sheet lead, 40 layers of asbestos millboard, and 20 sheets of galvanized iron, made up a block, 6 by 6 in. in area by $10\frac{3}{4}$ in. deep. It was tested for the full series of loads, ranging from 1 500 to 24 000 lb. on the scale-beam of the testing machine, which was equivalent to unit loads on the pads ranging from 35 to 450 lb. per sq. in.

Typical results from this series of tests are shown in Fig. 10. For a unit load of 450 lb. per sq. in., the average value of the damping ratio is $k = 0.48$, and, consequently, the percentage of damping for ten pads is 52. When the unit load was 340 lb. per sq. in., k was 0.45 and D was 55%; and with a unit load of 62 lb. per sq. in., the results were the same.

The average for the entire test series, comprising 228 vibrograph readings, and unit loads ranging from 35 to 450 lb. per sq. in., k, was found equal to 0.51, and, consequently, the total average damping percentage for ten pads was 49. The results indicate that as the unit load on the pad increases, the damping effect diminishes, which is confirmed by the results obtained with other materials comprised in this series of tests.

For a single pad of this type, of nominal thickness of 1 in., carrying a unit working load of 450 to 500 lb. per sq. in., and for the low frequencies covered by the tests (1 100 to 1 725 oscillations per min.), the percentage of amplitude of vibration damped out may be fixed at about 5.

EFFECT OF CONSTRAINT

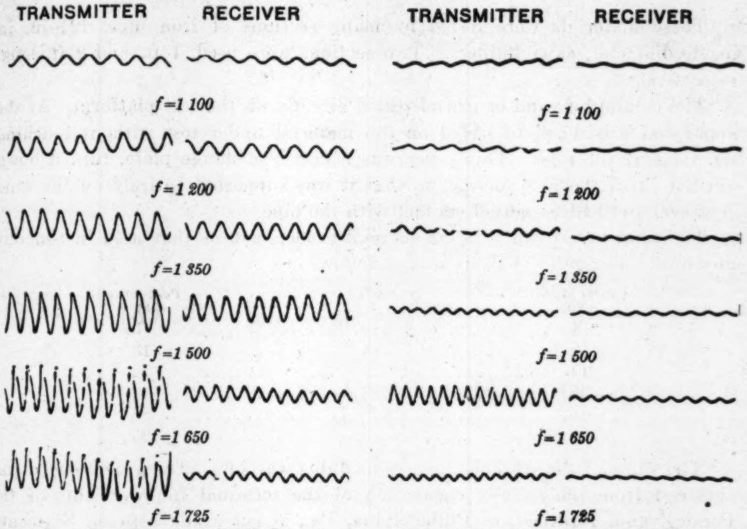
A series of vibrograph records was taken with the test pad replaced by a machined steel bar, 15 by $3\frac{1}{4}$ by $1\frac{1}{2}$ in. in size, placed on edge and crosswise, on the test platform; that is, with the $3\frac{1}{4}$-in. dimension vertical. The steel plate forming the test platform was also machined, and these were solidly clamped together. As might be anticipated, the vibrograph records taken under these conditions showed no damping effect.

When the clamps were removed, however, so that the bar was free from the plate, except for the load holding them together, the vibrograph records showed a notable damping effect. In Fig. 11(*a*) and Fig. 11(*b*), the load is 156 and 203 lb. per lin. in. of contact, respectively. This indicates that a non-rigid connection between two structural members transmitting load, equivalent to lack of constraint, has a pronounced effect in damping the transmission of vibration between these members. To verify this further, the machined steel bar was replaced by a knife-edge bearing, 4 in. long. The results for a light and a heavy load are shown in Fig. 11(*c*) and Fig. 11(*d*).

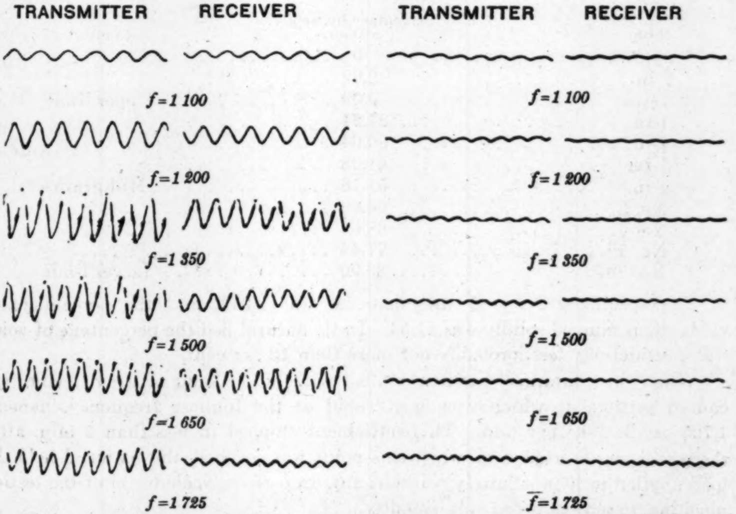
For the light load, the results shown in Fig. 11(*a*) and Fig. 11(*c*) are very similar. For the load of 16 250 lb., it is to be noted that the width of the H-beam, or receiver, was 8 in., and, therefore, the load per linear inch in Fig. 11(*b*) was 2 031 lb., whereas the knife-edge bearing was only 4 in. long, and, consequently, the load in Fig. 11(*d*) was 4 062 lb. per lin. in. of contact.

In Fig. 11(*d*) the amplitudes are very much less than in Fig. 11(*c*), due to the greater load and, consequently, smaller relative effect of the forced vibration. However, the ratios of amplitudes in both diagrams are almost exactly the same, giving an average damping percentage in both cases of 45. The actual numerical value of this damping factor, however, is not of special importance; the chief feature is that damping of a substantial amount results from lack of constraint.

This general principle, that lack of constraint damps the transmission of vibration, may be applied to great advantage in structural design in numerous ways. It also serves to explain the properties of sand and gravel as vibration dampers.



(a) Steel Bar, Free; Load 1 250 Pounds (b) Steel Bar, Free; Load 16 250 Pounds



(c) Knife-edge Bearing; Load 1 050 Pounds (d) Knife-edge Bearing; Load 16 250 Pounds

FIG. 11.—VIBROGRAPHS WITH SOLID STEEL PLATE AS INSULATION MEDIUM.

SAND AND GRAVEL

These materials were tested by using sections of iron pipe, 12¼ in. in inside diameter, as containers. Two sections were used, 1 ft. and 2 ft. long, respectively.

The column of sand or gravel rested directly on the test platform. At the upper end a cover-plate rested on the material under test without touching the sides of the pipe. This cover was a cast-iron flange plate, turned down until it fitted the pipe loosely, so that it was supported entirely on the sand or gravel and had no actual contact with the pipe.

The sand tested was of a commercial grade, such as that used in concrete mixtures. The sieve analysis was as follows:

Tyler sieve, No.	Percentage coarser.
4	5
8	13
14	22
28	43
48	83
100	95

The value of the Abrams fineness modulus was 2.65. The gravel tested was obtained from the subway excavation of the terminal improvements of the Pennsylvania Railroad, at Philadelphia, Pa., at sub-grade between Sixteenth and Seventeenth Streets. It was found to be glacial drift, ground up to all degrees of fineness. The sieve analysis was as follows:

Mesh.	Percentage by weight, coarser.	Limits.
2½-in.	0	......
2-in.	2.02	......
1½-in.	9.09	Upper limit
1-in.	24.24	......
¾-in.	30.64	......
½-in.	41.08	......
⅜-in.	51.18	Mid-grain
No. 3	56.90	......
No. 6	68.69	......
No. 10	77.44	......
No. 20	89.90	Lower limit

The specific gravity of this material was 2.373, and the percentage of voids when tamped solidly was 21.54. In its natural bed the percentage of voids was considerably less, probably not more than 12 per cent.

When the container was newly filled with either sand or gravel, vibration caused settlement which was most rapid at the highest frequency, namely, 1 725 oscillations per min. This settlement stopped in less than 5 min. after the motor was started, and, when this point was reached, the material held the load applied to it indefinitely, without any drop of the scale-beam of the testing machine, except as noted subsequently.

However, there was found to be considerable difference in stability between the sand and the gravel. When the container was filled with sand, and a cover-plate 12 in. in diameter was used, leaving a clearance of ⅛ in. all around between cover and pipe-wall, there was no settlement at the lower frequencies, but, at 1 725 oscillations per min., the sand began to rise through the crack and the cover-plate settled accordingly.

This did not happen with gravel. To test its stability under vibration, a cover-plate only 10 in. in diameter was tried, thus leaving a clearance of 1⅛ in. all around, between cover and pipe-walls. Although none of the material near the top of the container was larger than ½ to ¾ in. in diameter, it showed no tendency to rise through this wide opening or crack, and it was found possible to maintain a constant load at any frequency within the speed limits of the motor. Since the vibrograph records taken did not show any difference between sand and gravel as regards their properties as vibration dampers, the conclusion is that it is advantageous to use gravel rather than sand as subfoundation material because of its greater stability and because it is less liable to shift and settle under vibration.

As regards vibration properties, sand and gravel are essentially different from materials like cork, rubber, felt, and asbestos. The latter are all compressible and resilient, whereas sand and gravel are incompressible and nonresilient. Referring to the vibrograph records for cork, felt, and rubber (Figs. 3 and 7), it is evident that their action as vibration dampers, or, more properly, as insulators, is due to the fact that they act like a spring, permitting a certain dynamic freedom, or independence, between the two systems, without affecting the static load transmitted. Lacking such resilient properties, there is of course still the possibility that sand and gravel may absorb energy of vibration by internal work or frictional resistance between the particles. This possibility, therefore, must be examined.

It seems obvious that if there was sufficient motion between the particles of sand or gravel to absorb the energy of vibration by internal work, they would not show the stability they are known to possess. On this feature considerable information may be gained from seismology. On seismograph records, such as those taken at the U. S. Bureau of Standards, two sets of waves are clearly distinguishable. In fact, they form a characteristic feature of such records. When an earthquake is recorded on this instrument, there first appears a compression wave of relatively small amplitude, which is supposed to travel in practically a direct line from the disturbance through the earth's crust, that is, within a layer possibly 1 000 miles in depth. After a considerable interval of time a second wave suddenly makes its appearance, of much greater amplitude than the first. This is said to be a surface wave traveling close to the surface of the earth.

The amplitude of these waves is the important feature from the standpoint of the action of such materials as rock, soil, gravel, sand, etc., as vibration dampers. This may be illustrated from actual records taken on the seismograph at the U. S. Bureau of Standards, which has an amplification ratio of 1 000.

For example, the surface wave from an earthquake 2 000 miles distant, traced a graph 4 in. in width. The surface wave from an earthquake under the Pacific Ocean, 10 000 miles distant, traced a graph fully 1 in. in width. An earthquake in Bulgaria, 7 000 miles distant, traced a graph for the surface wave more than ¾ in. in width.

It would seem to be possible from a comparison of such records to study the damping effect of distance traveled on the amplitude of vibration, but apparently very little has been done along this line. Experimental data obtained during the World War on location of guns by means of soil vibration, indicated that vibration set up by gun-fire was observable for a distance of ¾ mile. In prospecting for oil, soundings are made by noting the time required for vibrations set up by the explosion of a depth charge to reach the surface, but practically no data are available as to the effect of distance traversed on damping the amplitude of vibration. It is sufficiently evident, however, from these and similar instances that within the limited distance of a few feet there can be no marked damping effect due to absorption of the energy of vibration by internal work. Whatever damping properties sand and gravel may possess, therefore, must be independent of the depth of the material within the actual limits usually met in engineering practice.

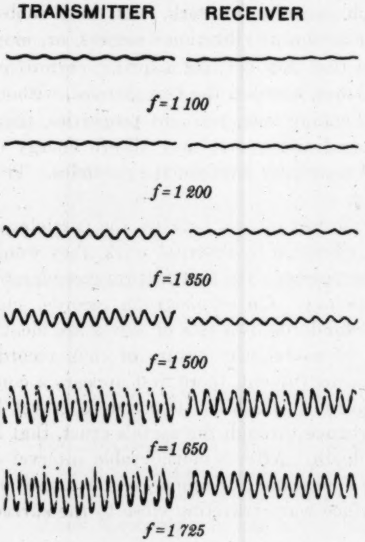


FIG. 12(*a*).—VIBROGRAPH OF 12-INCH LAYER OF SAND, LOADED TO 19 LB. PER SQ. IN.

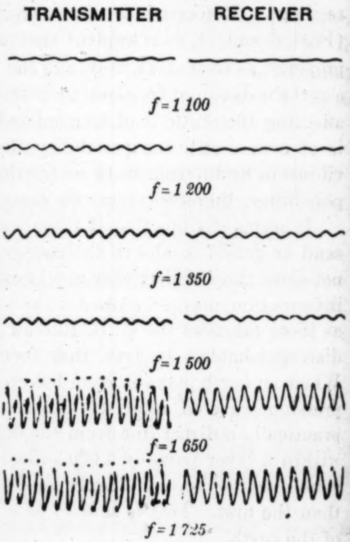


FIG. 12(*b*).—VIBROGRAPH OF 24-INCH LAYER OF SAND, LOADED TO 19 LB. PER SQ. IN.

In the course of this investigation, however, a large number of vibrograph records were taken with both sand and gravel. Typical results from these tests are shown in Figs. 12 and 13. The general conclusion is that such damping of vibration as results from the use of sand or gravel is a surface effect, similar to that which results from any looseness or lack of rigidity in a structural connection through which the forced vibration is transmitted. A practical instance of this effect may be noted along any railroad track in the motion of the ties on the ballast when a train passes. Such freedom damps the vibration at the source, localizing the disturbance instead of transmitting it elsewhere, as would be the case if the track were rigidly supported on a steel viaduct or solid masonry.

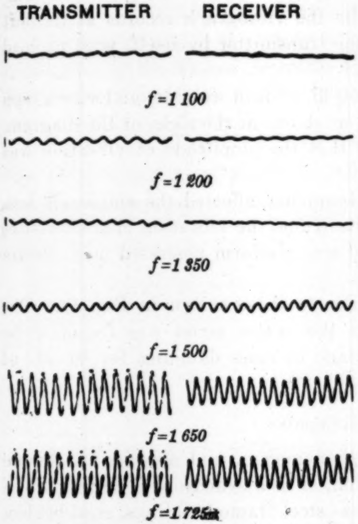


Fig 13.—Vibrographs of Gravel Sample, 24 In. Deep, Load, 19 Lb. per Sq. In.

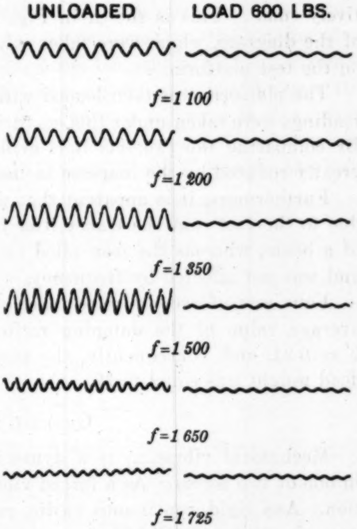


Fig. 14.—Vibrographs Showing Effect of Mass Damping.

Mass Damping

Mass may be used as an inertia damper to reduce the amplitude of a forced vibration. For example, in the case of a large Diesel engine, the bed-plate is usually so heavy that no vibration whatever can be felt while standing on it; yet at some point in a structure remote or near-by, there is often a serious vibration. Evidently, this was transmitted from the engine through the bed-plate, but the acceleration it produced in the relatively large mass was so small as to be imperceptible. It re-appears, however, as a large acceleration in some structural member of relatively small mass, the natural frequency of which happens to synchronize with the engine speed.

It may be noted, however, that when a massive foundation is used under machinery, although apparently it may be effective as a vibration damper, it is not a proper substitute for dynamic balancing because it is inefficient from the standpoint of power losses and strain on the frame of the machine.

With the apparatus used in these tests, the forced vibration was produced by a relatively small mass, namely, the unbalance of the pulley moving at a high acceleration. The radius of this "unbalance" was 1¾ in.; therefore, at the highest speed of 1 725 rev. per min. the centripetal acceleration was:

$$\frac{1.75}{12}\left(\frac{2\ \pi\ 1\ 725}{60}\right)^2 = 4\ 759 \text{ ft. per sec.}$$

The steel I-beams, motor, and test platform of which the test apparatus was made up, were relatively very heavy with respect to the "unbalance" on the pulley. The acceleration set up in these heavy members was, therefore, relatively small. This is shown in Fig. 14 by the vibrograph records at the left of the diagram, which were taken with the transmitter by itself, with no load on the test platform.

The platform was then loaded with 600 lb. of iron weights, and vibrograph readings were taken under this condition, as shown at the right of the diagram. By comparing the two sets it is evident that the amplitude of vibration was greatly reduced by the increase in mass.

Furthermore, it is apparent that the frequency affected the amplitude less, due to the fact that the transmitter by itself has the vibration characteristics of a beam, whereas the iron piled on the test platform possessed only inertia and was not affected by frequency.

Four sets of records were taken similar to those shown in Fig. 14. The average value of the damping ratio for the entire series was found to be $k = 0.21$, and, consequently, the percentage of mass damping for 600 lb. of dead weight was equal to 79.

GENERAL CONCLUSIONS

Mechanical vibration is a dynamic phenomenon, and appears in general in one of two forms: As a forced vibration, or as a synchronous or free vibration. Any rigid, continuous media, such as steel frame buildings, steel bridges or viaducts, or monolithic concrete structures, serve as excellent conductors of vibration, but no data are available as yet with respect to the relative conductivity of different types of construction as regards vibration.

To prevent the transmission of vibration, it should be "short-circuited" or confined to a limited region as close as possible to the source from which it originates.

For relatively light loads, such as machinery, there are many different media which may be used for isolation purposes. These depend on resilience for their insulating properties. None of them dissipates the energy of vibration by internal work or molecular friction to any appreciable extent.

For heavy loads, such as occur on piers and footings of tall buildings, the materials available for insulation are limited, and their resilience and damping properties are restricted to relatively narrow limits.

Synchronous vibration may be prevented entirely or eliminated by such modification in the design of the part affected as shall throw its natural frequency out of step with the exciting frequency.

Sand and gravel when used as insulators do not absorb the energy of vibration by internal friction; they owe such effectiveness as they possess to their lack of rigidity. Their insulating property is, therefore, largely a surface effect.

Vibrations of high frequency and relatively small amplitude are the most objectionable from a physiological standpoint.

These tests and such other data as are available, indicate that valuable results may be anticipated from research on the effect of modifying structural detail with a view to damping vibration without recourse to resilient insulators, especially as relates to vibration arising from subway operation and street traffic.

ACKNOWLEDGMENTS

The tests on which this study is based, were performed in the Engineering Laboratory of the Towne Scientific School of the University of Pennsylvania. The research facilities of this laboratory were made available through the courtesy of H. C. Berry, M. Am. Soc. C. E. The hair felt used in these tests was a product especially prepared for machinery insulation and was obtained through the courtesy of Mitchells, Ashworth, Stanfield, and Company, Limited, of London, England.

DISCUSSION

J. P. Den Hartog,[5] Esq. (by letter).—The paper deals with a subject of great interest, on which little research has been done. The general problem of vibrations in structures as well as in machinery is quite complicated, as Mr. Slocum remarks, and it is too often discussed in a purely empirical manner. It is possible, however, to give a perfectly rational explanation of many of these phenomena, including that which is the subject of this paper.

The author states that "no practical solution has yet been found of the vibration characteristics of such a primary element as a simple beam of uniform cross-section under load." This is not entirely true, since Rayleigh's method[6] gives, with very little calculation, a solution which is at least 95% correct; it is a "practical solution." If one wants to put some more work into the problem, Ritz's method[6] can be used, giving a solution in a series form to any desired accuracy. Both these methods are extensively used by vibration engineers.

The experimental set-up of Fig. 1 can also be completely calculated if the full data concerning the masses, flexibilities, and damping constants of the various members are known.

These items are not given in the paper. The elements that are least known in this set-up are the "load from the testing machine" and the flexibility of the supports between the I-beams and the platform.

The "load from the testing machine" consists partly of mass or dead weight and partly of spring pressure. These two causes have the same effect in compressing the test pad; but they behave radically different, dynamically. From the results in Fig. 2 (*a*), however, certain tentative conclusions can be drawn.

In the first place, the curve in Fig. 4 with the data in the description of Fig. 2 (*a*) makes it possible to calculate the spring constant of the test pad, which is 3 000 lb. per in. of deflection. This is so flexible as compared with the H-beam, that the latter flexibility can be safely neglected. The system of the test pad, H-beam, and testing-machine head then can be represented by Fig. 15.

Let k_1 (= 3 000 lb. per in.) represent the spring of the test pad, while M and k_2 are the equivalent mass and spring caused by the H-beam and the testing-machine head. If this system is subjected to a vibratory motion, $a \sin \omega t$, at its bottom end, the mass, M, has a motion, $b \sin \omega t$, in which,

$$b = a \times \frac{k_1}{k_1 + k_2} \; \frac{\omega_n^2}{\omega_n^2 - \omega^2} \quad \ldots\ldots\ldots\ldots (3)$$

Equation (3) is the principal formula of the spring vibration absorber. In this formula, $\omega_n^2 = \dfrac{(k_1 + k_2)}{M}$, is the natural frequency of the system.[7]

---

[5] Research Dept., Westinghouse Elec. & Mfg. Co., East Pittsburgh, Pa.

[6] A description of Rayleigh's as well as of Ritz's method can be found in "Vibration Problems in Engineering," by S. Timoshenko, D. Van Nostrand Co.

[7] The complete theory can be found in the *Electric Journal*, Vol. XXI (1924), p. 160, in an article by C. R. Soderberg and, also, in the book by Timoshenko previously noted, p. 33.

The ratio, $\frac{b}{a}$, of the amplitudes of the "receiver" and the "transmitter", as calculated from Equation (3), has been plotted in Fig. 16. The curve shows that for very slow motions ($\omega = 0$) the ratio, $\frac{b}{a}$, is equal to $\frac{k_1}{(k_1 + k_2)}$; that is, the "static" condition. At a certain speed, ($\omega = \omega_n$), resonance occurs and the motion becomes infinite theoretically. For very fast motions of the bottom end ($\omega = \infty$), the mass, M, is too inert to follow it, and its amplitude is zero.

It is seen that the vibration absorber works only when its impressed frequency is high with respect to its natural frequency, which is another way of stating that a soft spring is needed. It is seen that for a stiff spring the absorber may become injurious; that is, the received vibration may be larger than the transmitted vibration. The test results indicate that they were conducted at the extreme right of the diagram of Fig. 16. In fact, it is possible that in the present test set-up there is some low running speed in which the amplitudes received are larger than those transmitted.

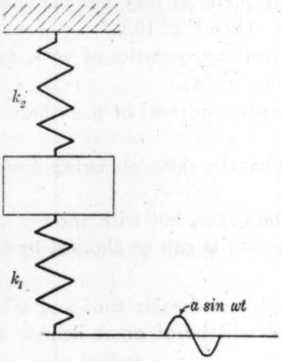


FIG. 15.—SCHEMATIC ARRANGEMENT OF TEST PAD, H-BEAM, AND TESTING MACHINE HEAD.

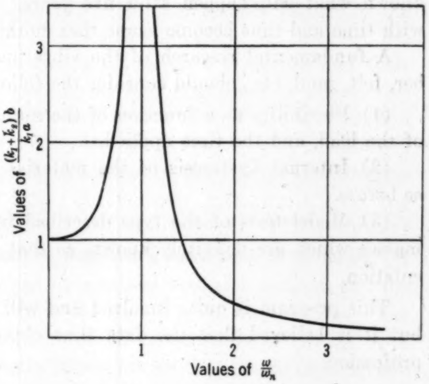


FIG. 16.—CURVE SHOWING THE RATIO, $\frac{b}{a}$, OF THE AMPLITUDE OF THE RECEIVER TO THAT OF THE TRANSMITTER AS A FUNCTION OF THE FREQUENCY OF THE INPUT VIBRATION.

In case the unbalanced motor had been placed on the H-beam directly above the test pad, while replacing the equivalent dead weight of the motor in its original location on the I-beams, the vibration of the I-beams would still be larger than that of the H-beam (at speeds from 1 100 to 1 500 rev. per min.). In this case, again, the result would be a "negative damping".

If, instead of corkboard, steel springs of the same flexibility had been used, the results would have been very much the same, since the tests were taken for points that do not coincide with the critical speed of the system in Fig. 15, so that internal friction of the material is of no importance.

The resonant frequency of the I-beams cannot be appreciably affected by the remainder of the system since the flexibility of the test pad is so great. A rough calculation of this frequency from the available data indicates a higher frequency than the records show. A possible explanation for this is that the supports between the I-beams and the test platform have some flexibility, or that the test platform itself is not entirely steady.

When the corkboard test sample is replaced by sand or gravel, which have a higher modulus of elasticity, the k_1 of Equation (3) becomes larger and, consequently, the condition is displaced toward the left in Fig. 16, thereby showing a larger transmitted vibration.

Summarizing, it can be stated that proper vibration absorption can be accomplished fundamentally by supporting the vibrating system on such springs that the natural frequency of the system becomes low with respect to the frequency of vibrations. This can be done by steel springs or by organic materials like cork, rubber, etc.

The writer has had some experience with these absorbers as applied to machinery, and generally has used steel springs, for the reason that present knowledge concerning organic materials is insufficient. It is quite possible to mount a structure on corkboard and get good immediate results, but it is not known what will happen after five years. The material may become stiffer with time and thus become worse than nothing. (See Fig. 16.)

A fundamental research of the vibration-absorbing qualities of cork, rubber, felt, sand, etc., should consider the following points:

(1) Flexibility as a function of the number of repetitions of the vibration, of the load, and the time applied.

(2) Internal hysteresis of the material under the same changing factors as before.

(3) Model tests of the type described in the paper, but with springs and masses which are definitely known, so that the results can be checked by calculation.

This program is quite involved and will take considerable time and effort, but it is believed that the data thus obtained will be of great help to the profession.

Francis G. Tatnall,[8] **Esq.** (by letter).—The subject of vibration is of importance as a new field of engineering and, by this time, should have its specialists and a quantity of experimental knowledge. This does not seem to be the case, however, because little definite information on vibration is available and few specialists seem to be in the field.

Vibration studies are a specialist's job, because even though frequency and amplitude of vibration can be fairly well catalogued for any structure, the more difficult question is encountered: "What is to be done about it?"

Damping and insulating are not the answers. These appear to be only makeshifts, usable until the time comes when vibration can be measured and predicted so that it can be cared for by proper design. The completed structure, or parts of it that can vibrate as a whole, must be designed so that their natural frequency will conveniently avoid any forced vibration that will tune in to set up resonance.

---

[8] Mgr., Test Equipment Div., Southwark Foundry & Machine Co., Philadelphia, Pa.

With existing apparatus the frequency and amplitude of vibration can be measured under actual working conditions. With this knowledge, the machine or structure under consideration can be re-designed, and then must be tested again to see whether its natural frequency is high enough or low enough to keep out of step with external forced vibration. There is too much "cut-and-try" in this method so that ninety-nine out of a hundred times it is still a laboratory procedure. In some few instances, this method is actually applied to commercial design.

The paper dealt with low frequency, large amplitude vibration. The method is a good laboratory test for comparing vibration dampers. Very much higher frequencies, taken from structures in the field under actual operating conditions, can be measured with existing apparatus. Furthermore, vibration can be recorded from a large number of points simultaneously. The double resistor type of electric telemeter has been frequently used for such purpose and to good effect.[9] This instrument is attached to the member to be investigated and vibration in the member changes the resistances of a pair of highly refined carbon resistors which proportionately unbalances a Wheatstone bridge. An oscillograph, connected across the apex of the bridge, records faithfully the frequency and amplitude of vibration. If twelve telemeters, for instance, are attached by very small cap screws to twelve points on a structure and their lead wires brought together at a panel board and connected to the twelve elements of a single oscillograph recorder, twelve simultaneous vibration records can be made on a single 12-in. film, and the frequency can be obtained by comparison with a thirteenth record. This record may be that of 60-cycle alternating current, a break-circuit Swiss chronometer operating a reflecting kicker, or a special tuning-fork marking off 0.01-sec. intervals. The equipment is first calibrated so that known values of amplitude have been established. A magnification of four hundred times can be obtained with this method, and the amplitude of vibration is measured from the actual wave on the film. The principle of measuring electrical unbalance on so sensitive a circuit as a Wheatstone bridge eliminates all inertia effects except that of the oscillograph vibrator which is infinitely small.

Frequencies up to 1 200 cycles per sec. can be recorded at the sensitivities requisite for this work, although frequencies as high as 8 000 cycles per sec. have been recorded. The frequency is limited by the natural period of the oscillograph vibrator. As sensitivity in recording increases, frequency decreases. In other words, the greater the magnification of amplitude, the less the frequencies that can be recorded. Most vibrations encountered in commercial engineering structures are less than 300 cycles per sec., so this permits high magnification. Errors in recording range from 3 to 10%, depending on the severity of vibration.

Perhaps the telemeter can furnish additional information on the subject of vibration. Perhaps it can extend the work of Mr. Slocum, so as to record the actual damping effect, in structures under service conditions, of the materials he has studied in the laboratory. It can surely pick out the resonance or synchronisms as illustrated in Fig. 17.[10] When resonance is attained, the oscillograph trace can be seen to rush to it as a peak and becomes fainter (see position marked 10th harmonic, 1 650 rev. per min., top line, Fig. 17). The heavy regular wave is the timing trace. The superimposed wave is the forced vibration in the part. When the natural frequency of the part is not approached this wave is regular and well defined, but when synchronism is reached, it flies all to pieces. The figures, 10th, 12th, 15th, etc., designate the harmonics, and the speeds noted are the rates of forced vibration.

---

[9] See "Recent Developments and Applications of the Electric Telemeter," by O. S. Peters, *Technologic Paper No. 247*, U. S. Bureau of Standards.

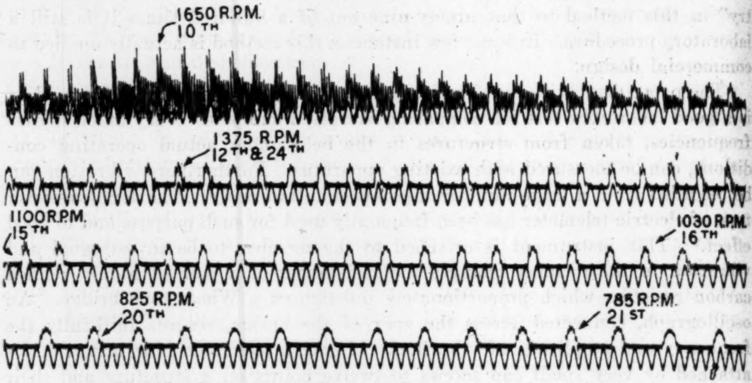


FIG. 17.—OSCILLOGRAPH SHOWING RESONANCE OR SYNCHRONISM.

By means of the telemeter, vibration can be measured, recorded, studied, and catalogued, but it must be remembered that there the function of the telemeter stops. What remedies to use to overcome the phenomenon is another problem entirely.

W. M. DAWLEY,[11] M. AM. SOC. C. E. (by letter).—A consideration of some of the fundamental equations of motion as derived by Mr. J. W. S. Rayleigh[12] leads one to question whether the arrangement of the test apparatus adopted by Mr. Slocum was the best for the purpose (see Fig. 1). The test pad at the center and the unbalanced rotor near the quarter-point of the transmitter I-beam with the plane of rotation at right angles to the length of the beams, would set up a harmonic overtone vibration in the beams having a period one octave above the normal period of the undamped beam, the latter period being more or less damped out. The test pad, therefore, rests at the nodal point in so far as the horizontal vibrations are concerned and does not respond fully to the impressed vibrations.

The test load applied to the one-third point of the H-beam acts as a damper to produce a node at that point. On the assumption that the flexibility of the test pad permits horizontal vibrations of that end of the H-beam resting on it, the period would be one octave above the normal period or "gravest node" of vibration of the beam with both ends fixed, but otherwise free.

---

[10] As recorded by the Cleveland Wire Spring Co.

[11] Engr., Land and Tax Dept., Erie R. R., Cleveland, Ohio.

[12] "Theory of Sound," by J. W. S. Rayleigh, Second Edition, 1894, reprinted 1926.

In discussing this phase of the problem, Messrs. J. Ormondroyd and J. P. Den Hartog, have shown that a simple steel bar, 2 in. by ¾ in. by 60 in., lying flat and simply supported at the ends and supporting a 26-lb. direct-current motor at its center with two 8-lb. weights, one attached at the quarter-point, and the other at the three-fourths point, of the length, had natural frequencies of 750 and 2 800 rev. per min.[13]

It would seem to have simplified the interpretation of results if the rotor, test pad, and load from the testing machine had all been placed in the same vertical plane. This plane would pass through the test pad at right angles to the beams, with the center of the rotor on a level with the steel plate under the test pad for horizontal motion tests, and vertically under the test pad for vertical motion tests.

The use of such an unbalanced rotor as that described by the author introduces undesirable variations which confuse the analysis. Up to a certain critical speed the motor armature, with its unbalanced pulley, will rotate about its geometric center and the force that produces vibration will be only that of the unbalanced component, a few ounces or pounds. At speeds greater than this critical speed the vibration will involve the whole mass of the armature and pulley as well. The tendency then is for the entire rotating part to revolve about the common center of gravity of armature and unbalanced weight, which is not the geometric center of the shaft. Since the unbalanced pulley is at one end of the motor shaft, that end of the motor will vibrate more than the opposite end at high speeds, thus tending to produce a rotary motion similar to that of a rotating pendulum, except that the axis of rotation is horizontal instead of vertical.

There is also the further complication that the phase of the vibration at speeds greater than the critical speed is almost, but not exactly, opposite the phase at speeds less than the critical and that the amplitude of vibration changes at the critical speed, being less at the higher speeds, as shown in Fig. 14. The effect of one or several unbalanced weights rotating in different parallel planes has been described at considerable length by Mr. R. Eksergian[14] in connection with the problem of dynamically balancing the reciprocating and rotating parts of a locomotive.

Viewed from another angle, consider the H-beam and the testing-machine load as a steady mass which it is desired to set in motion by means of the unbalanced weight on the motor pulley, and that the amplitude of the receiver graphs is a fair measure of the energy that has survived the trip through the transmitter I-beams and test pad.

There is not much variation in the energy transmitted under the conditions illustrated by Figs. 2, 7, 9, 10, 11(*b*), and 11(*d*). That is, approximately the same percentage of the energy developed by the unbalanced rotor is transmitted notwithstanding the fact that the more or less flexible test pads permitted the transmitter I-beams and plate to vibrate more or less freely. In other words, the amplitude of the vibrations set up in the transmitter beams is not the proper function to compare with the amplitude of receiver graphs to determine the efficiency percentage of the insulator pads.

---

[13] *Transactions*, Am. Soc. Mech. Engrs.; Applied Mechanics, Vol. 50, Paper 7; abstract in *Mechanical Engineering*, December, 1928, p. 975.

[14] *Transactions*, Am. Soc. Mech. Engrs., Vol. 51, No. 17; Railroads, Vol. 51, No. 5.

The flexible I-beams interposed between the rotor and the steel plate on which the test pad rests should be eliminated, or the energy lost in setting the transmitter beams in vibratory motion should be ascertained by comparing the amplitude of the unloaded transmitter with that of the transmitter with the test pad and testing machine load applied.

However, transmitter graphs for $f = 1\,100$ shown in Fig. 7(*a*) and Fig. 7(*b*), seem to be inconsistent with this latter point of view, because the damped vibration of the lightly loaded transmitter shows a greater amplitude than the unloaded transmitter graph for the same speed shown in Fig. 14.

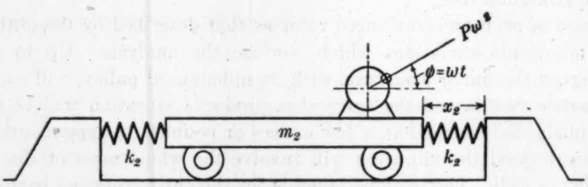


FIG. 18.

In the vibration research laboratory at Leland Stanford, Jr., University the shaking table[15] consists of a heavy platform mounted on four street-car wheels, and anchored by springs (in compression) to heavy concrete piers (see Fig. 18). The platform is made of 8-in. **H**-beams bolted and welded together; its dimensions are 10 by 12 ft. with a weight, m_2, of approximately 6 000 lb. In order to reduce the friction of the table motion to a minimum, the street-car wheels have been ground concentrically with their axles and mounted on ball bearings.

In considering the type of motion to be imparted to this table it was decided that it must be "a civilized" disturbance, a motion of the table that can be repeated again and again with great accuracy; and, if possible, it must be a motion that can be treated mathematically with some degree of comfort, in order that the accelerations and velocities involved may be known quite accurately.

These considerations have led to the adoption of two distinct types of linear motions, namely, a damped, free vibration of the table, resulting from an elastic impact of a pendulum; and a continuous, forced vibration, set up by a revolving unbalanced fly-wheel mounted on the shaking table. The two types of resultant motion are, of course, quite different. In the case of the pendulum, maximum acceleration occurs when the initial displacement is quite small and the succeeding maxima of the accelerations are each less than the preceding, the difference between the first and second being considerably greater than the difference between the succeeding maxima. The maxima of the accelerations are at maximum amplitudes. The damping of the free vibration is a combination of constant damping and velocity damping.

---

[15] See Lydik S. Jacobsen, in *Bulletin*, Seismological Soc. of America, Vol. 19, No. 1, March, 1929, Fig. 18.

In the case of the unbalanced pulley the resultant vibration of the table is very nearly a simple harmonic motion or sine curve.

The equation of motion is:

$$m_2 x_2'' + c_2 x_2' + k_2 x_2 = P \omega^2 \cos \omega t \ldots\ldots\ldots\ldots\ldots (4)$$

Integrated, this equation becomes:

$$x_2 = \frac{P \omega^2 \cos (\omega t - \psi)}{[c_2^2 \omega^2 + (k_2 - m_2 \omega^2)^2]^{\frac{1}{2}}} \ldots\ldots\ldots\ldots\ldots (5)$$

in which,

ψ = arc tan $\dfrac{c_2 \omega}{k_2 - m_2 \omega^2}$;

m_2 = mass of table;

k_2 = characteristic of anchor springs;

x_2 = displacement of table;

c_2 = damping factor;

P = the unbalanced moment;

ω = the angular velocity, in radians per second; and

t = time, in seconds.

When resonance occurs, $\omega = \sqrt{\dfrac{k_2}{m_2}}$, and the displacement of the table is,

$$x_2 = \frac{-P \omega \sin \omega t}{c_2} \ldots\ldots\ldots\ldots\ldots (6)$$

The pulley is mounted on the table independently of the motor, and, judging from the photographic illustration, it appears to be 2 ft. or more in diameter. It is loaded with eight pigs of lead braced under the rim and a shot tank located between the spokes, which can be filled with any desired amount of shot and emptied without stopping the rotation, thus varying the amplitude at will by altering the amount of unbalance without changing the speed. The motor is also set on the table and is belt-connected with the pulley.

Fig. 19[16] illustrates quite clearly how the maximum amplitude of vibration of the shaking table, which simulates Mr. Slocum's unloaded transmitter, varies with the speed of rotation of the unbalanced pulley.

It would seem then that the best measure of the value of a "vibration damper or insulator" would be its capacity to absorb or dissipate the kinetic energy of the applied forced vibrations.

To measure this capacity a wave of known kinetic energy should be applied to one side of the material under test and the surviving energy of the same wave emerging from the opposite side should be measured by suitable means. For simplicity in analysis the form of the impressed wave should be simple harmonic motion, although it may be possible to analyze the resultant record of two concurrent harmonic motions of different periods, amplitudes, and epochs, provided the period, amplitude, and phase of either one is known or is capable of independent determination.

Experiments conducted by the writer in June, 1925, at East Binghamton, N. Y., may be of some value in considering the subject of vibrations set up in and transmitted through the surface of the earth at no great depth, since they show to some extent the damping effect of distance traveled, on amplitude. These tests were made to determine the increase in amplitude of vibrations in a greenhouse, due to moving the main track of the Delaware, Lackawanna and Western Railroad from a location 122 ft. away to a parallel location 80 ft. away.

---

[16] See Lydik S. Jacobsen in *Bulletin*, Seismological Soc. of America, Vol. 19, No. 1, March, 1929, Fig. 16.

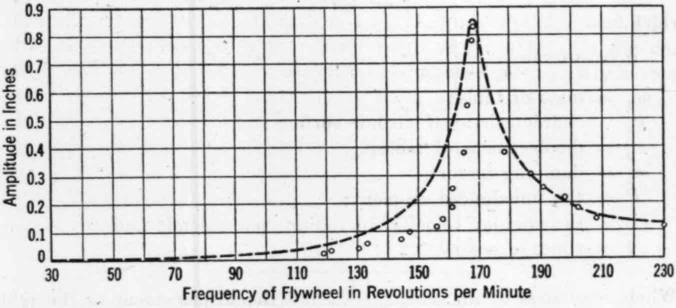


FIG. 19.—THEORETICAL CURVE FOR VISCOUS DAMPING, PROPORTIONAL TO x'_2 ACCORDING TO L. S. JACOBSEN.

The U. S. Topographical Survey map for this region shows the location to be at the narrowest part of the Susquehanna River Valley, the hills rising on either side of the narrow river bottom to heights of from 1 500 to 1 600 ft. The elevation of the greenhouse floor and adjoining field is 840 ft., and the normal surface of water in the river, 831 ft. above mean sea level.

This topographical layout would appear to preclude the possibility of there being any soft or yielding material underlying the sandy loam soil, mixed with the rather large pebbles on which the railroad embankment and greenhouse are situated. The probability is that it is gravelly loam, then sand, and then gravel down to hardpan, clay, and boulders mixed, and, finally, bedrock.

The instrument used in making the tests consists of a pendulum of 93.5 lb. (42.2 kg.) weight suspended by piano wire from an engineer's transit tripod. It has an undamped period of 2.222 sec., or 27 complete oscillations per min. For portability the weight is made of four accurately turned cast-iron disks, each 8¾ in. in diameter and 1¾ in. thick, mounted in two pairs on a 1-in. steel bolt which passes through an accurately bored hole in the center of each. The bolt has a taper of a few thousandths of an inch, so that each disk fits tightly. The upper pair of disks is separated ½ in. from the lower pair by three brass studs to permit two thrust arms, at right angles to each other, to be attached to the center of the steel bolt at the center of percussion of the pendulum. The entire arrangement is securely clamped together on the bolt by means of a nut countersunk in the top disk. By means of these thrust arms and a system of multiplying levers attached to the steel bedplate which is in contact with the earth, the relative motion of the earth and pendulum is magnified 148.6 times.

Record was made on smoked paper passing over a brass cylindrical drum by a fine steel needle-point in the extremity of each of the recording levers. One of the levers is bent at right angles, in order to record the two rectangular co-ordinates of motion on the same smoked sheet at a distance of 3 in. apart. To reduce the friction to a minimum all connections between thrust arms, multiplying levers, and recording levers are made either by springs or by polished conical steel pivots in V-shaped white sapphire jewels.

All moving parts are of duraluminum or aluminum. The weight of the moving parts of the amplifying lever system for one component of motion is 12⅛ grains, or about 1/3200 part of that of the pendulum.

Anticipating, from the work of Mr. Prince, of the Transit Commission in the Times Square case of the New York Subway, that the period of vibrations set up by trains would be of the order of 20 per sec., or more, all moving parts were kept as light in weight as was consistent with rigidity to resist deformation of the individual parts at high frequencies.

Record was taken of vibrations set up in the surface of the earth on the cultivated earth floor inside the greenhouse. Oak stakes 3 in. square were driven 36 in. into the ground with the tops a few inches above the surface, and the earth was thoroughly compacted about the stakes with a sledge hammer. Three "set-ups" were made, as shown in Fig. 20. The records cover the passing of fifty-eight trains of all classes—passenger, freight, milk, and combination—in both directions, east and west; and they include five days of operation.

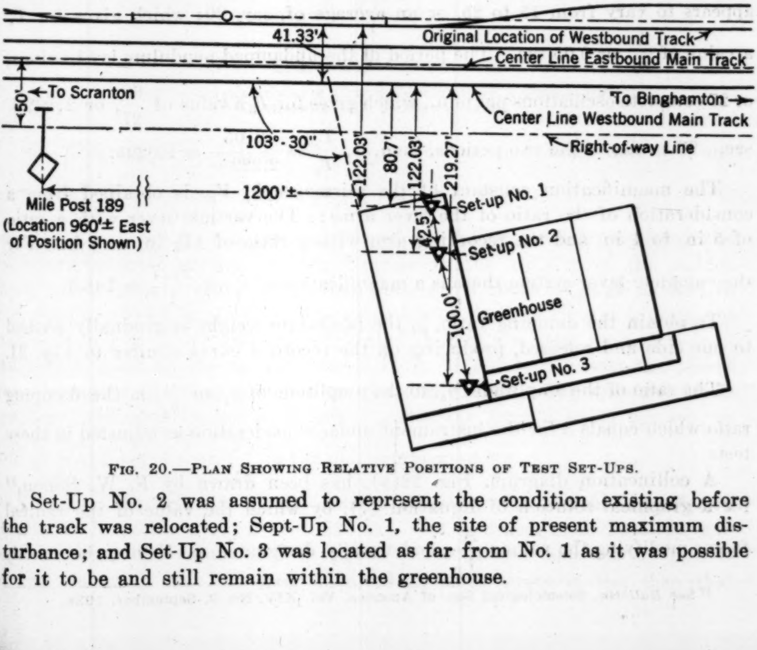

41.33'
Original Location of Westbound Track
Center Line Eastbound Main Track
50'
To Scranton
To Binghamton
Center Line Westbound Main Track
103°-30"
122.03'
80.7'
122.03'
219.27'
Right-of-way Line
1200'±
Mile Post 189
(Location 960'± East
of Position Shown)
Set-up No. 1
42.5'
Set-up No. 2
100.0'
Greenhouse
Set-up No. 3

FIG. 20.—PLAN SHOWING RELATIVE POSITIONS OF TEST SET-UPS.

Set-Up No. 2 was assumed to represent the condition existing before the track was relocated; Sept-Up No. 1, the site of present maximum disturbance; and Set-Up No. 3 was located as far from No. 1 as it was possible for it to be and still remain within the greenhouse.

The method of mounting the seismograph on stakes driven in the ground was adopted to harmonize with the type of construction used for the greenhouse, which consists of a glass roof supported on iron pipe posts resting on short wooden fence posts driven in the ground flush with the surface.

The maximum amplitude recorded for each train movement at each set-up was taken, for comparison, as representing a measure of the maximum earth movement to which the column footings in that vicinity were subject.

Interpretation and Tabulation of Results.—In any seismograph constructed on the pendulum principle with a system of multiplying levers, the actual amplitude of the earth's motion may be calculated from the amplitude of the trace on the seismogram by means of the formula:

$$A_e = \frac{A'}{V_0}\sqrt{\left(1+\frac{T_e^2}{T_0^2}\right)^2 - 4\,\frac{\dfrac{T_e^2}{T_0^2}}{1+\dfrac{1}{\pi^2}(\log_e \rho)^2}} = \frac{A'}{V_0}\,U \ldots\ldots\ldots (7)$$

in which,

A_e = the amplitude of the earth's motion from zero position;
A' = the amplitude of the trace record from zero position;
T_e = the period of the earth movement, in seconds;
T_0 = the period of the undamped pendulum of the seismograph;
V_0 = the magnification constant of the seismograph; and,
ρ = the damping ratio.

From the records the number of complete earth vibrations per second appears to vary from 15 to 25, or an average of, say, 20; which gives for T_e a value of $\frac{1}{20}$, or 0.05 sec. The period of the undamped pendulum is 54 swings, or 27 complete oscillations per min., which gives for T_0 a value of $\frac{60}{27}$, or 2.222+ sec. The ratio of the two periods, then, is, $\frac{T_e}{T_0} = \frac{0.05}{2.222+} = 0.0225$.

The magnification constant of the seismograph, V_0, is obtained from a consideration of the ratio of the lever arms: The vertical lever with a ratio of 5 in. to 1 in. and the recording arm with a ratio of 14½ in. to ½ in. For the complete lever system there is a magnification of $\frac{5}{1} \times \frac{29.7}{1} = 148.5$.

To obtain the damping ratio, ρ, the pendulum weight is gradually pushed to one side and released, producing on the record a curve similar to Fig. 21. The ratio of the amplitude, y_1, to the amplitude of y_2, or $\frac{y_1}{y_2}$, is the damping ratio which equals 3 for the instrument under consideration as adjusted in these tests.

A collineation diagram, Fig. 22(*a*), has been drawn by F. W. Sohon,[17] for a graphical solution of Equation (7), by which the value of the radical factor modifying the value, $\frac{A'}{V_0}$, and usually designated as U, may be readily obtained. This diagram consists of a circle of unit radius calibrated for the period ratio, $\frac{T_e}{T_0}$, a tangent line calibrated for values of the damping ratio, ρ, and a secant line calibrated for the value of the radical factor, U.

---

[17] See *Bulletin*, Seismological Soc. of America, Vol. XIV, No. 3, September, 1924.

Laying a straight-edge from the point on the circle indicated by the ratio of $\frac{T_e}{T_0} = 0.0225$, which is very near the point of "sudden shock" at the junction of the circle with the secant line, to the point on the tangent indicated by the damping ratio of 3, the value of U is read at the intersecting point on the secant line.

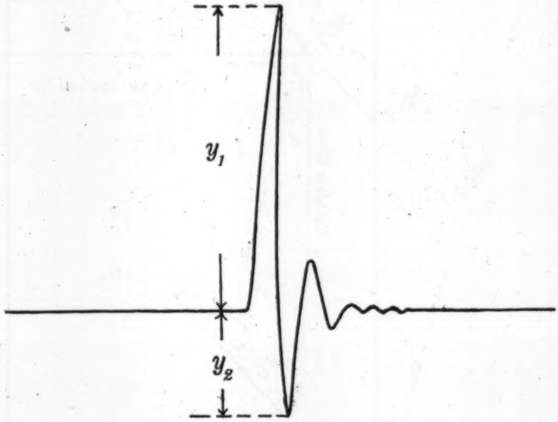


FIG. 21.—CHARACTERISTIC CURVE PRODUCED BY A PENDULUM TO DETERMINE THE DAMPING RATIO.

The diagram (Fig. 22), however, is on such a small scale that it is not convenient to use where values of $\frac{T_e}{T_0}$ are less than 0.1, or where earth periods, T_e, exceed 4 or 5 per sec.; therefore, the writer has computed and calibrated a small portion of this diagram in the vicinity of the sudden shock point, using a radius more than sixty-seven times as large as that in Fig. 22(a). (See Fig. 22(b).) The direction of the line from u to ρ can be obtained on Fig. 22(a) and a line parallel thereto can then be drawn from the proper point on the enlarged circle (Fig. 22(b)) to intersect the secant line for values of U as indicated by the dashed line. The enlarged circle has been calibrated for earth periods, T_e, running from 3 to 25 per sec., as well as for values of u or $\frac{T_e}{T_0}$ in the neighborhood of 0.1.

To show the relation of the smaller diagram, the dashed line, drawn parallel with the line from $u = 0.0225$ to $\rho = 3$, is drawn from $T_e = 20$, and cuts the secant for values of U at 0.9996 or 0.0004 less than unity. This is so near to unity that the error involved in using unity is less than that

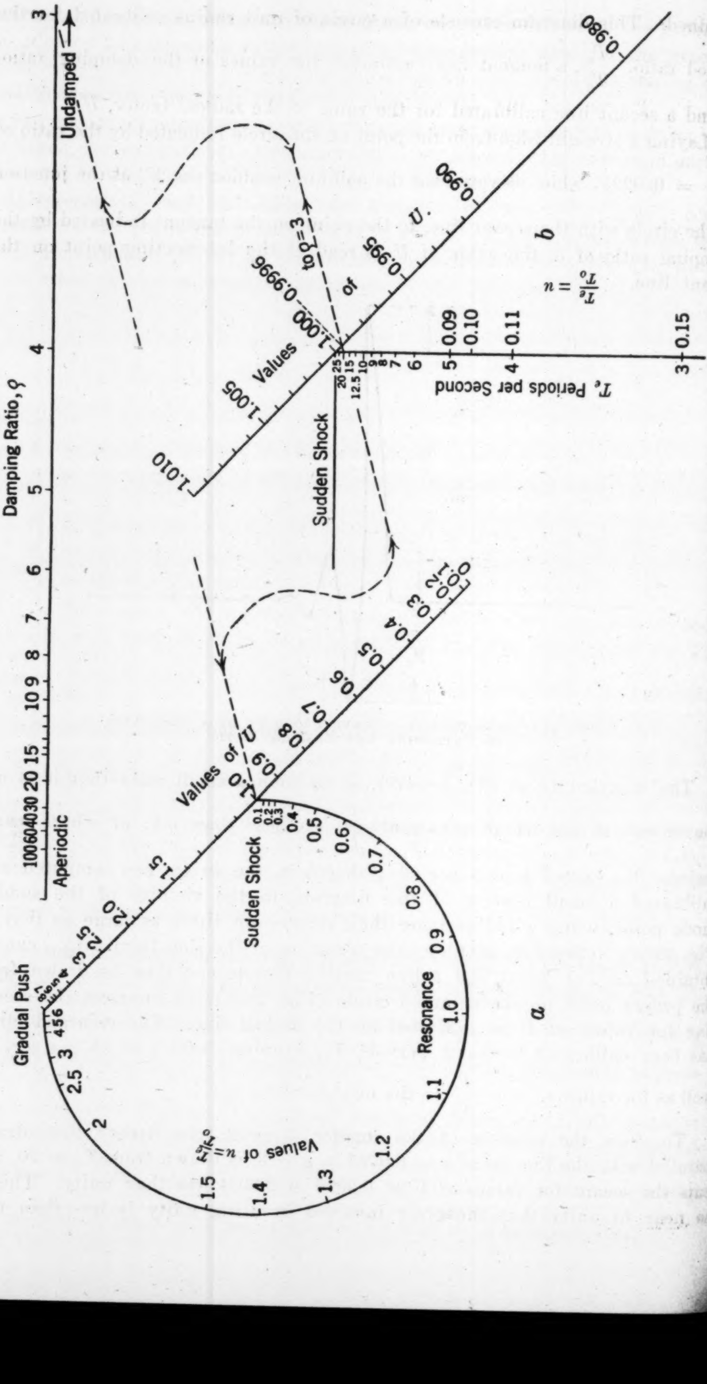

Damping Ratio, ρ
100 60 40 30 20 15 10 9 8 7 6 5 4 3
Aperiodic
Undamped
Gradual Push
Sudden Shock
Resonance
Values of $n = \frac{T_e}{T_o}$
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4 1.5 2 2.5 3 4 5 6
Values of U
1.0 0.9 0.8 0.7 0.6 0.5 0.4 0.3 0.2 0.1 0.0
1.5 2.0 2.5 3.5
a
Values of U
1.010 1.005 1.000 0.9996 0.995 0.990 0.980
to ρ=3
Sudden Shock
T_e Periods per Second
25 20 15 12.5 10 9 8 7 6 5 4 3
0.09 0.10 0.11 0.15
$n = \frac{T_e}{T_o}$
b

FIG. 22.—DIAGRAM FOR THE SOLUTION OF EQUATION (7).

TABLE 1.—VIBRATION RECORDS OBSERVED AT GREENHOUSE. LOCATIONS AS SHOWN IN FIG. 20.

Record No.	Class of train.	Direction.	Speed, in miles per hour.	Weight of engine, in tons.	Number of cars.	Weight of cars, in tons.	Total weight, in tons.	AMPLITUDE, A', IN INCHES, AS SCALED FROM RECORD TRACE.		COMPUTED MOMENTS OF THE EARTH, A_e, IN INCHES.	
								Motions parallel to main track.	Motions at right angles to main track.	Motions parallel to main track.	Motions at right angles to main track.
SET-UP NO. 1; OBSERVED JUNE 20, 1925.											
1	Passenger.........	Eastbound	....	242	10	536	778	0.097	0.132	0.00066	0.00091
2	Passenger.........	Westbound	....	224	4	161	385	0.119	0.385	0.00082	0.00281
3	Passenger.........	Eastbound	58.0	226	3	170	396	0.077	0.155	0.00053	0.00107
4	Passenger.........	Eastbound	20.7	226	7	242	468	0.045	0.139	0.00031	0.00096
5	Milk...............	Westbound	40.0	235	26	847	1 082	0.090	0.403	0.00062	0.00278
6	Milk...............	Westbound	60.0	235	22	692	927	0.292	0.372	0.00201	0.00256
7	Passenger.........	Westbound	31.6	240	6	199	439	0.152	0.446	0.00105	0.00307
8	Passenger.........	Eastbound	36.0	242	9	561	803	0.086	0.202	0.00059	0.00139
9	Passenger.........	Westbound	42.9	225	4	208	433	0.092	0.383	0.00063	0.00264
10	Milk...............	Eastbound	30.0	235	25	817	1 052	0.066	0.114	0.00045	0.00078
11	Milk...............	Eastbound	18.3	235	22	710	945	0.259	0.470	0.00178	0.00324
12	Passenger.........	Westbound	60.0	242	10	628	870	0.058	0.083	0.00040	0.00057
13	Freight............	Westbound	18.0	150	6	170	320	0.053	0.161	0.00036	0.00111
14	Freight............	Westbound	24.0	240	35	865	1 105	0.083	0.176	0.00057	0.00121
Totals....................										0.01078	0.02460
Average....................										0.00077	0.00176

TABLE 1.—(*Continued.*)

Record No.	Class of train.	Direction.	Speed, in miles per hour.	Weight of engine, in tons.	Number of cars.	Weight of cars, in tons.	Total weight, in tons.	AMPLITUDE, A', IN INCHES, AS SCALED FROM RECORD TRACE.		COMPUTED MOMENTS OF THE EARTH, A_e, IN INCHES.	
								Motions parallel to main track.	Motions at right angles to main track.	Motions parallel to main track.	Motions at right angles to main track.
SET-UP NO. 1; OBSERVED JUNE 23, 1925.											
1	Passenger.........	Eastbound	43.9	242	9	531	778	0.173	0.220	0.00119	0.00152
2	Passenger.........	Westbound	48.6	235	4	165	400	0.136	0.282	0.00094	0.00194
3	Passenger.........	Eastbound	41.9	226	5	291	517	0.090	0.171	0.00061	0.00118
4	Passenger.........	Eastbound	18.5	235	6	203	438	0.071	0.114	0.00049	0.00079
5	Milk...............	Westbound	36.0	235	26	849	1 084	0.202	0.431	0.00139	0.00297
6	Milk...............	Westbound	34.6	240	23	714	954	0.222	0.314	0.00153	0.00216
7	Combination	Westbound	50.0	242	6	193	435	0.127	0.242	0.00088	0.00167
8	Passenger.........	Eastbound	39.1	240	9	576	816	0.178	0.197	0.00123	0.00136
9	Passenger.........	Westbound	40.9	225	3	158	383	0.188	0.287	0.00129	0.00198
10	Milk...............	Eastbound	30.0	235	27	886	1 121	0.256	0.235	0.00176	0.00162
11	Milk...............	Eastbound	27.7	240	21	687	927	0.138	0.102	0.00095	0.00070
12	Passenger.........	Westbound	50.0	242	8	511	753	0.349	0.418	0.00291	0.00288
13*	Passenger.........	Westbound	51.4	242	4	160	402	0.257	0.393	0.00177	0.00271
14*	Passenger.........	Westbound	45.0	242	9	569	811	0.179	0.290	0.00123	0.00200
15	Light..............	Eastbound	....	250	..	.....	250	0.084	0.143	0.00058	0.00099
16	Freight............	Westbound	Stopping	228	53	1 860	2 088	0.220	0.439	0.00151	0.00303
Totals....................										0.01976	0.02950
Average of Set-Up No. 1, June 23, 1925....................										= 0.00124	0.00184
Average of Set-Up No. 1, June 20, 1925....................										= 0.00077	0.00176
Total....................										= 0.00201	0.00360
Average of all trains recorded at Set-Up No. 1....................										= 0.00101	0.00180

* Record taken June 22, 1925.

TABLE 1.—(*Continued.*)

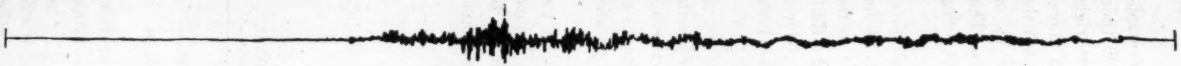
Record No.	Class of train.	Direction.	Speed, in miles per hour.	Weight of engine, in tons.	Number of cars.	Weight of cars, in tons.	Total weight, in tons.	AMPLITUDE, A', IN INCHES, AS SCALED FROM RECORD TRACE.		COMPUTED MOMENTS OF THE EARTH, A_e, IN INCHES.	
								Motions parallel to main track.	Motions at right angles to main track.	Motions parallel to main track.	Motions at right angles to main track.
SET-UP NO. 2; OBSERVED JUNE 21, 1925											
1	Passenger..........	Eastbound	42.3	242	10	531	773	0.061	0.121	0.00042	0.00083
2	Passenger..........	Westbound	54.5	235	4	150	385	0.141	0.296	0.00097	0.00204
4	Passenger..........	Eastbound	38.7	225	6	193	418	0.076	0.102	0.00052	0.00070
5	Milk................	Westbound	50.0	235	26	847	1 082	0.109	0.154	0.00075	0.00106
6	Milk................	Westbound	24.0	235	22	690	925	0.040	0.063	0.00028	0.00043
7	Combination	Westbound	30.0	224	5	144	368	0.105	0.267	0.00072	0.00184
8	Passenger..........	Eastbound	39.1	242	10	636	878	0.059	0.106	0.00041	0.00073
10	Milk................	Eastbound	33.3	235	26	843	1 078	0.043	0.063	0.00030	0.00043
11	Milk................	Eastbound	32.7	235	21	688	923	0.066	0.171	0.00045	0.00118
12	Passenger..........	Westbound	46.2	242	10	624	866	0.134	0.284	0.00092	0.00196
15	Freight............	Westbound	....	228	57	1 715	1 943	0.143	0.297	0.00099	0.00205
16*	Freight............	Eastbound	15.3	287	68	2 936	3 223	0.052	0.120	0.00036	0.00083
17	Freight............	Eastbound	....	287	69	2 953	3 240	0.067	0.096	0.00046	0.00066
Totals....................										0.00755	0.01464
Average....................										0.00054	0.00105

* Less than one-half of train on record.

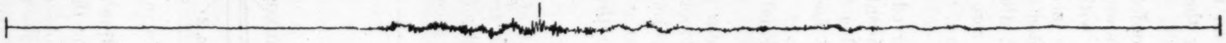
TABLE 1.—(*Continued.*)

Record No.	Class of train.	Direction.	Speed, in miles per hour.	Weight of engine, in tons.	Number of cars.	Weight of cars, in tons.	Total weight, in tons.	AMPLITUDE, A', IN INCHES, AS SCALED FROM RECORD TRACE.		COMPUTED MOMENTS OF THE EARTH, A_e, IN INCHES.	
								Motions parallel to main track.	Motions at right angles to main track.	Motions parallel to main track.	Motions at right angles to main track.
SET-UP NO. 3; OBSERVED JUNE 22, 1925											
2	Passenger.........	Westbound	52.9	241	4	155	396	0.059	0.082	0.00041	0.00056
3	Passenger.........	Eastbound	48.6	226	3	170	396	0.056	0.053	0.00037	0.00036
4	Passenger.........	Eastbound	18.2	235	6	2 000	435	0.035	0.033	0.00024	0.00023
5*	Milk	Westbound	30.0	241	26	843	1 084	0.084	0.087	0.00058	0.00060
6	Milk	Westbound	29.5	241	24	747	988	0.080	0.118	0.00055	0.00081
7	Combination.......	Westbound	43.9	242	6	197	439	0.063	0.064	0.00043	0.00044
8	Passenger.........	Eastbound	34.6	242	9	577	819	0.055	0.045	0.00038	0.00031
9	Passenger.........	Westbound	37.5	225	4	210	435	0.050	0.065	0.00034	0.00045
10	Milk	Eastbound	26.1	241	25	817	1 058	0.030	0.036	0.00021	0.00025
11	Milk	Eastbound	25.3	241	21	687	928	0.033	0.043	0.00023	0.00030
12	Passenger.........	Westbound	52.9	242	8	511	753	0.049	0.233	0.00034	0.00161
15	Freight............	Eastbound	14.5	228	40	1 539	1 767	0.037	0.038	0.00025	0.00026
Totals..........										0.00433	0.00618
Average..........										0.00036	0.000515

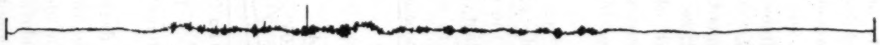
*Coasting, not using steam.



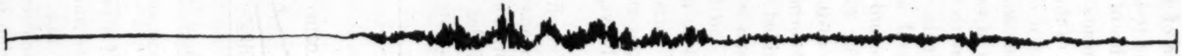
(*a*) SET-UP NUMBER 1; AMPLITUDES PARALLEL TO THE MAIN LINE



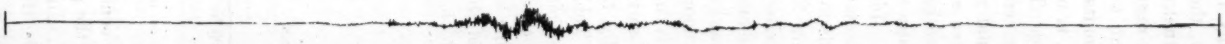
(*b*) SET-UP NUMBER 2; AMPLITUDES PARALLEL TO THE MAIN LINE



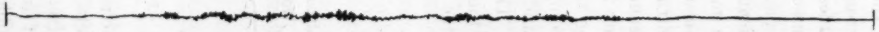
(*c*) SET-UP NUMBER 3; AMPLITUDES PARALLEL TO THE MAIN LINE



(*d*) SET-UP NUMBER 1; AMPLITUDES AT 90° WITH MAIN LINE



(*e*) SET-UP NUMBER 2; AMPLITUDES AT 90° WITH MAIN LINE



(*f*) SET-UP NUMBER 3; AMPLITUDES AT 90° WITH MAIN LINE

FIG. 23.—COMPARISON OF TRACES FOR RECORD NO. 8, SHOWING AMPLITUDES PARALLEL AND AT RIGHT ANGLES TO THE MAIN LINE TRACKS.

which is unavoidably involved in scaling the amplitude on the record, and this value of unity, therefore, has been used in reducing the results of the writer's tests to actual amplitude of the earth's motion.

In reducing the results of the tests, the maximum recorded double amplitude or displacement, to both sides of the zero position, shown for each train and for both horizontal components of motion, parallel with and at right angles to the track, is measured to the nearest 0.001 in. by means of a micrometer caliper gauge. The distance thus obtained is divided by 2 to get the amplitude from the zero position, and this amplitude, A', divided by V_0, or 148.6, gives the actual amplitude of earth movement from zero position. This computation has been carried out to the fifth decimal place, the result being in hundred thousandths of an inch.

Table 1 illustrates the general arrangement of observed data obtained during these tests. Identical record numbers apply to identical trains for each set-up. For example, in Fig. 23, Record No. 8 (Table 1) refers to the amplitudes observed during the eastbound passage of the Lackawanna Railroad Passenger Train No. 6 as determined on successive days. As reproduced in the paper, the trace is magnified 119.6 times the original vibration; the magnification of the original graph was 148.5.

The maximum amplitude of earth movement from zero position, A_e, is found usually, but not invariably, at the time the locomotive is opposite the instrument, the maximum parallel with the track seldom occurring at the same instant as that at right angles thereto. Motion at right angles to the track is generally, but not always, considerably greater than that parallel with the track (see Table 1).

A comparison of maximum amplitudes given in Table 1 shows that the westbound trains produce a movement nearly twice that of the eastbound trains. This damping of the vibrations from the eastbound track seems to be due to the weight of the 7-ft. cinder fill, ballast, and track of the westbound main line under which the vibrations passed to reach the seismograph.

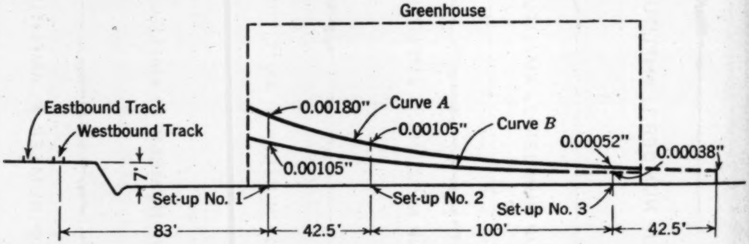


FIG. 24.—PROFILE OF OBSERVED AMPLITUDE AT RIGHT ANGLES TO THE MAIN LINE.

Conclusion.—In conclusion, if a profile is drawn on a line extending from the tracks through the greenhouse and passing through the three observation points (see Fig. 20), using the average of all the maximum observed amplitudes at right angles to the track at each set-up as elevations above this line, it will be possible to visualize the relation between amplitude of the earth's motion and distance from the track. Such a profile is illustrated by Curve *A* in Fig. 24.

Due to the position of Set-Up No. 2 that portion of the profile beyond it will represent the condition as to vibration existing prior to the relocation of the main tracks 41.33 ft. closer to the greenhouse.

If this part of the profile with a northward extension of 42.5 ft., based on the indicated decrease in amplitude with distance, is moved that distance south within the first profile so that Set-Up No. 2 coincides with Set-Up No. 1, then the difference between the gradient lines of the two profiles (Curve *A* minus Curve *B*) will represent the increase in amplitude affecting the greenhouse due to the shift in position of the main tracks.

The base line of the profile makes a northeasterly angle of 103° 30′ with the center line of the westbound main track, and is parallel with the center line of the greenhouse. The distances to the several points along this line from the center line of the westbound main track are, therefore, correspondingly greater than the right-angle distances to the same points as given in Fig. 20.

A record was also taken of an Erie Railroad freight train passing on the opposite side of the Susquehanna River, at a distance of about ½ mile from Set-Up No. 1. The maximum observed amplitude parallel with the track was 0.00013 in. and that at right angles, 0.00011 in.

Time marks on the records were spaced 10 sec. apart. The instant when the locomotive was opposite the seismograph is indicated in Fig. 23 by a freehand mark above the record. It was observed that vibrations began from 4 to 6 sec. before this instant.

STEPHEN E. SLOCUM, M. AM. SOC. C. E. (by letter).—In the discussion of this paper by Mr. Den Hartog the point is made that there is a rational explanation of the phenomena accompanying these tests, which of course is the case. There is a wide distinction, however, between a qualitative explanation and a quantitative measurement, and in actual practice it is undeniably true that the determination of nodes, natural frequencies, etc., is, in general, best made by experiment. The idea which the writer intended to convey was that even in the simplest cases it is necessary to make certain assumptions as to uniformity of material and conditions of constraint before a numerical solution of the vibration characteristics can be attempted. In most actual structures, the constraints are so involved as to make any attempt at a theoretical solution impossible. For instance, it would seem to be a hopeless task actually to calculate the vibration characteristics of a modern rotary printing press, such as that used in printing the daily papers; or to determine by calculation at just what point in a skyscraper, and to what extent, traffic vibration would set up a synchronous disturbance.

Steel springs, mentioned by Mr. Den Hartog, can be used only in a few very special cases. The writer experimented extensively with steel springs some years ago in connection with engine mounting in automotive apparatus, and found that, to cushion the torque recoil, a very weak spring was needed, which was not stiff enough to resist road shocks; consequently, this type of mounting was found to be impractical. In railroad and street-car work, steel springs are used to lessen the impact from road shocks which tend to set up vibration, but to dampen the residual vibration actually set up, the remedy generally adopted is to separate the non-rigid metallic contacts by the insertion of pads of resilient damping material. For the special purpose for which this research was undertaken, namely, the determination of the physical properties and vibration characteristics of damping materials suitable for use as isolation pads under grillage foundations of tall buildings, steel springs are obviously out of the question.

In Mr. Tatnall's discussion, the principal point made is that change in design is a more effective remedy for vibration than isolation. This is true only of machinery built from a stock design and operated under fixed and uniform conditions. For instance, the crank shaft of a Diesel engine should be designed so as to be free from excessive torsional vibration at the operating speed of the motor. In machinery like compressors and blowers, however, it is usually impractical to eliminate vibration by changes in design, and the only feasible method of preventing the transmission of vibration is isolation from adjacent structures.

Mr. Dawley's criticism of the author's experimental set-up would be pertinent and valuable if the experiments had been intended as a basis for mathematical analysis of the compound system, although his suggestions by no means cover all the difficulties involved. As this series of experiments was apparently the first of its kind, at least so far as published results are concerned, there were numerous fundamental questions to be answered before the problem could be stated in sufficiently exact terms to warrant an elaboration of set-up which would differentiate results more closely.

For instance, some of the primary questions involved and on which no information could be gained elsewhere, were such as the following:

(*a*) For a substance like asbestos, what effect does increasing the unit load on the material have on its properties as a vibration damper?

(*b*) For lateral insulation of footings would a mastic joint be as effective as corkboard, or if the latter was used, would a layer 4 in. or more in thickness be necessary, or would a 1-in. layer be sufficient?

(*c*) For track insulation, would a layer of sand be more effective than a layer of gravel? This question was of prime importance, since if experiment had indicated that sand was greatly superior, its use would have involved excavating a layer of natural glacial drift, easily plowed up into gravel, and replacing it with sand.

Evidently none of these problems involved a mathematical determination of nodes or harmonics.

Mr. Dawley mentions that loading the H-beam at the third point would create a node at this point. This was exactly what was intended, as the effect of applying the load from the testing machine at a node was to cut out the testing machine from the vibrating system, and to just this extent reduce the complexity of the problem.

Mr. Dawley expresses as his opinion that the best measure of a vibration damper or insulator is its capacity to "absorb" or "dissipate" the kinetic energy of vibration. As a matter of fact, no pad of whatever material either absorbs or dissipates the energy of vibration. The only way in which this could occur would be through generating heat by molecular friction, and there is not the slightest evidence that anything of the kind occurs. The word which most fully describes a vibration damper is "isolation". The effect of such isolation is to short-circuit the vibration, reflecting a certain fraction of its energy back into the transmitting system; and to just the extent to which it is effective in reflecting the energy of vibration, it is successful in preventing its transmission to the receiving system.

To handle the vibration problem intelligently requires considerable versatility of method. In the case of machinery, the first consideration is whether change in design is possible, and if so, whether it will reduce vibration to a point where it is not objectionable. For heavy machinery, the design of the foundation is often involved in the problem. In the case of traffic vibration, the street pavement or track roadbed is an important factor to be considered. In tall buildings, an important remedy lies in isolation of the structure by various means. A notable consequence of the tests summarized in this paper was the discovery that a simple compressive pad was by no means the most effective method of applying resilient dampers. The tests also disclosed the fact that the range of damping materials commercially available is extremely limited, and that none possesses all the required properties of an effective damper or isolation medium under all conditions of use and loading.

As pointed out, vibration engineering should be a fertile field for the activities of the Society. Although there seems to be many specialists working independently in this field along such widely divergent lines as seismology in oil fields, and impact stresses in bridges, there is no attempt to co-ordinate their efforts or systematize their results. In London, England, the serious effects of traffic vibration on such notable structures as the Nelson Column, the National Gallery, in Trafalgar Square, St. Paul's Cathedral, and Westminster Abbey, has forced the question of remedying matters on the attention of the Office of Works, and this has made the question of vibration a permanent subject of research and discussion by the Science Committee of the Royal Institute of British Architects.

In Germany, the Scientific Council of the Vereins Deutscher Ingenieure took steps as early as 1923 to foster research in vibration engineering, and on December 11, 1924, that Society organized a Committee on Vibration. In order to give direction to this research, the Committee called attention to three specific problems, namely, dynamic strain in machines and structures, due to high frequency vibrations; general disturbance, such as noise and tremor due to vibration; and research in the technique of mechanical vibration, both in scientific laboratories and in industry, with the related problem of the endurance of materials of construction subject to vibration.

In 1927 this Committee on Vibration held a two-day convention at Braunschweig. The principal topics discussed at this conference were approved methods of conducting vibration experiments; laboratory methods in current use; necessary and sufficient conditions for judging the value of vibration measurements; energy absorption or damping capacity of building materials;

acoustic experiments with various materials and types of construction; structure of crystals as related to vibration endurance; and electrical apparatus for measuring the strength of sound.

In 1928, this Committee held a three-day session in Darmstadt, the principal topics considered being the fatigue of materials of construction subject to vibration; experimental measurement of vibration in railway tracks and bridges; natural modes of vibration of machine foundations; destructive effects of vibration due to street traffic as related to condition of pavement and type of tire; development of instrumental methods of measuring vibration; absorption of energy of vibration by materials of construction; and the use of annealed instead of hardened steel for machine springs.

For the first time, in the 1927 Edition of "Hütte", a well-known handbook for German engineers, a section was devoted to the subject of vibration.

Heretofore, the problem of vibration has been confined in its engineering aspects almost entirely to machine design and operation. In the United States the rapid increase in height of skyscrapers, together with a growing volume of street and subway traffic, has created a new and vital vibration problem. The rapid growth of this field in other lines, such as radio engineering and seismographic prospecting for oil on the Gulf Coast, has given a new aspect to the entire question. It has long been recognized that vibration is a basic phenomenon in physics and chemistry. It is equally apparent that it is a basic principle in many branches of technology, such as electrical engineering, sound reproduction, and geo-physics. The particular phase which commands immediate attention is the destructive effects of vibration on men and materials. In this particular line of effort, perhaps greater advances have been made in this country than elsewhere, but it still remains for American engineers to co-ordinate this accumulated knowledge and to systematize the principles of vibration control into a specialized branch of engineering. The Society is the obvious agency for this purpose.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1766

HIGH DAMS
A SYMPOSIUM[1]

PAGE

Past Experience with High Dams and Outlook for the Future.
By A. J. Wiley, M. Am. Soc. C. E. 130

Classification, Selection, and Adaptation of High Dams.
By D. C. Henny, M. Am. Soc. C. E. 139

High Dams: The Viewpoint of the Geologist.
By F. L. Ransome, Esq. 149

Construction Methods and Plant Layout at Coolidge Dam, in Arizona.
By J. G. Tripp, M. Am. Soc. C. E. 159

With Discussion by Messrs. F. G. Switzer, Fred A. Noetzli, Edward Godfrey, B. F. Jakobsen, Allen Hazen, H. de B. Parsons, P. Wilhelm Werner, C. A. P. Turner, F. W. Hanna, H. F. Dunham, Lars R. Jorgensen, William P. Creager, L. F. Harza, A. Floris, Joseph Wright, A. J. Wiley, D. C. Henny, and J. G. Tripp.

---

[1] Presented at the Joint Meeting of the Irrigation and Construction Divisions, San Diego, Calif., October 4, 1928.

PAST EXPERIENCE WITH HIGH DAMS AND OUTLOOK FOR THE FUTURE

By A. J. Wiley,[2] M. Am. Soc. C. E.

Synopsis

This is a brief account of past practice in the design of high dams, with comments that indicate that the higher dams of the future will be of a solid masonry gravity type, either arched or straight. As between a straight gravity dam and a dam of the same section arched in plan, preference is given to the latter, or the so-called "arched gravity dam".

Between an arched gravity dam in which—because of the preponderance of gravity action—the arch stresses are low, and a less-than-gravity section arch in which the arch action predominates, preference is given the arched gravity type. Reasons for the writer's belief that the arched gravity dam is more stable than a straight gravity dam of the same section are given in the paper. Furthermore, some suggestions for the design of future high dams are offered for consideration.

Comparison of Types

Dams can be divided into two classes, the earth embankment type and the masonry type. In the former the material is devoid of strength and prevents the passage of water only by the sum of the resistances of its individual particles. In the latter the inherent strength of the masonry forms an impenetrable barrier to the water. The strength of the earth embankment types is not subject to definite analysis. The mass is always greatly in excess of that required to resist the static pressure of the water, and is determined largely by previous experience as necessary to prevent percolation or seepage.

In the masonry types only a very small part of the mass is needed for watertightness and stability is subject to close analysis and mathematical determination.

Ordinary earth is probably less subject to the agencies that cause disintegration and decay than any other material of construction, and within its limits, the earth-fill dam is probably the most enduring of all types. The limiting condition of the earth dam and of its types—the hydraulic-fill and the rock-fill—is in the difficulty of making it impenetrable to water and in preventing its saturation and consequent instability. Experience up to the present indicates that earth embankments, hydraulic fills, and rock fills, although admirably adapted to many conditions for heights as great as 200 or 300 ft., are not likely to be adopted for dams of really great heights and large volumes of storage.

---

[2] Cons. Hydr. Engr., Boise, Idaho.

Masonry dams have no such limiting conditions and can be safely designed for any height likely to be needed in the future. On the other hand, masonry dams have one very rigid limiting condition from which earth dams are free; that is, the necessity of hard and durable rock foundations. This is particularly true of arched dams which require not only very hard foundations, but may need even harder abutments, because the arch thrust may be greater than the cantilever reaction.

Dams of Masonry.—Since their introduction masonry dams have always been given the preference for structures of great height, but the application of scientific principles to design only dates back to the development of the gravity type of masonry dam in about 1855.

The first high gravity dams were straight in plan and were built of stone masonry laid in hydraulic lime mortar. Prior to 1870 it was assumed that uplift could not act on the base, and no allowance was made for it in the design of these dams; nor was any attempt made either to reduce or prevent it.

The design of high dams since about 1910 has been influenced by the practically complete replacement of stone masonry by concrete, and by the rapid extension of the use of reinforced concrete. There has thus been developed at about the same time (1) the arched gravity dam of comparatively long radius; (2) various types of arch dams of shorter radius and less-than-gravity section; (3) multiple-arch dams; (4) buttress type, or Ambursen dams; and (5), the multiple-dome dam.

Gravity dams have been either straight or arched. It would probably be correct to state that the use of arched gravity dams has been largely confined to the Western States, and that this is due to the more rugged topography which gives deep and narrow canyons where high dams of short radius can be used.

Arched Gravity Dams.—In the design of arched gravity dams the same cross-section is used as in a straight gravity type, on the assumption that the reduction in gravity action, due to the convergency of the radii, would be more than offset by horizontal arch action. In the earlier designs of arched gravity dams the same practice of ignoring uplift was followed as had been the custom in straight gravity dams. Many important dams of the arched gravity type—such as the Cheesman Dam, 235 ft. high, and the Roosevelt Dam, 280 ft. high—were built on this theory. These two dams were not only among the last high, arched, gravity dams to be built of stone masonry, but they were about the last in which no attempt was made to provide for uplift in the design or to remove it.

In nearly all the arched gravity dams built of concrete, uplift is reduced by deep cut-off trenches at the up-stream face, and by pressure grouting and drainage of the foundation. Uplift in the body of the dam is guarded against by vertical drains near the up-stream face. Examples of this type are the Arrowrock Dam, 350 ft. high above the foundation and 260 ft. above the stream bed; the Don Pedro Dam, 278 ft. above the stream bed; the Exchequer Dam, 325 ft. above the stream bed; the Lancha Plana Dam, 350 ft. high; and the Owyhee Dam, which will have a height of 325 ft. above the stream bed, and 405 ft. above the foundation.

In all these and in many other dams of similar design, the convergency of the radii is assumed to be offset by arch action and the uplift reduced, by pressure grouting and drainage, to an amount which can be taken care of by arch action.

High Arched Dams.—At the same time as the arched gravity dams, various types of less-than-gravity arch dams with shorter radii, were being built in narrow canyon sites. The arched dams have been either of fixed radius, variable radius, or variable radius with constant-angle types.

In arched dams of constant radius the section is a reduced gravity type with the up-stream face vertical. The aim is to reduce the vertical tension and compression in the up-stream and down-stream faces to a minimum on the assumption that tension in the one and excessive compression in the other will be avoided as in the full gravity section dam.

In the variable radius designs the aim is to carry the greatest possible part of the load on the arch, although the tendency of this is to increase the vertical tension and compression at the faces. The variable radius with constant angle, or the "constant angle arch", is a variable radius type in which the volume is reduced to the minimum by maintaining a constant central angle of about 120 degrees.

Examples of the fixed radius type are: The Shoshone Dam, 250 ft. above the stream bed and 330 ft. above the foundation, with a base of 142 ft.; the Salmon River Dam, 220 ft. high, with a base of 101 ft.; the Warmsprings Dam of the overflow type, 100 ft. high, with a base of 27 ft.; the Gibraltar Dam, 185 ft. high, with a base of 65 ft.; and the Melones Dam, of the overflow type, 200 ft. high and 85 ft. thick at the base.

An example of the variable radius type is the Gibson Dam, 195 ft. high, with a base of 87 ft. Many dams of the variable radius or constant angle type have also been built, including the Kerchoff (overflow) Dam, 105 ft. high, with a base of 36 ft.; the Salmon Creek Dam, 168 ft. high, with a base of 38 ft.; the Cushman Dam, 280 ft. high, with a base of 52 ft.; and the Pacoima Dam, recently completed, which is 375 ft. high, with a base of 96 ft.

The principle of the multiple-arch dam is very old, but the present type, with inclined up-stream face, has been developed since about 1910. Examples of multiple-arch dams are the Gem Lake Dam, 112 ft. high; the Lake Hodges Dam, 136 ft. high; the Florence Lake Dam, 160 ft. high; the Palmdale Dam, 175 ft. high; and the Tirso Dam, in Italy, 239 ft. above the foundations.

Multiple Dome Dam.—The multiple dome dam is a new type of which the only example is the Coolidge Dam on the Gila River, in Arizona. It may be described as a multiple-arch dam with great length of span and very massive piers. By thickening these piers toward the base the arch radius is shortened progressively from the top to the bottom of the dam. The arches as well as the piers are very massive so that the thin construction of the multiple-arch type is avoided.

The Coolidge Dam is 250 ft. high and consists of three arches and two piers, the sides of the canyon forming the buttress for the arches at each end of the dam.

Ambursen Type Dams.—In the Ambursen dam, the arches connecting the piers or buttresses of the multiple-arch type are replaced by reinforced concrete flat slabs or beams. Many dams of this type have been built since about 1905. The latest are the Stony Gorge, 120 ft. above the stream bed, and the Rodriquez Dam, 140 ft. above the stream bed and 200 ft. above the foundations, which is now (1928) under construction.

Higher Dams of the Future.—The highest dam yet undertaken is the Owyhee Dam (now under construction), which is 405 ft. high above the regular foundations and 520 ft. above the deepest point of cut-off. The San Gabriel Dam, 500 ft. high, is now under construction. The Boulder Canyon Dam, 700 ft. high above the foundations, is now authorized for construction.

The San Gabriel Dam is arched to a 1 400-ft. radius and is of a gravity design with an increased section to allow for convergence of the radii. The Owyhee Dam is of the arched gravity type with a 500-ft. radius and no increase of section to allow for convergence of radii.

There will be, no doubt, a constant improvement in the design of all types of dams and an increase in the height to which they can safely be built, but at present the arched gravity type of masonry dam is slightly in the lead with respect to height.

SELECTION OF TYPE

Arch vs. Straight Section.—In places where an arch of such a radius as will give any degree of arch action can be used, it should always be given the preference over a straight dam of the same section. The length of radius that will give useful arch action is subject to wide differences of opinion. It is the writer's view that, assuming the construction to be done in such a way as to give vertical joints practically free from leakage, an arch of any required radius is preferable to a straight dam. This is based on the fact that if there are no open joints extending entirely through it, the slightest deflection of the dam as a gravity cantilever will cause an equal deflection in the horizontal arch and a division of the load in some proportion between the cantilever and the arch.

If there are open joints extending through the dam, that are water-tight at the down-stream face as assumed, then there is a reduction of the water pressure on the up-stream face from the back pressure on the joints. To the writer there seems to be a culpable waste in not utilizing as much as possible the horizontal compressive strength of the concrete in the arch, along with its vertical compressive strength in the cantilever.

Arched Gravity Type.—As between an arched gravity dam and an arched dam of considerably less than gravity section and correspondingly shorter radius, preference in the case of a very high dam should always be given to the arched gravity type. In the case of the arched gravity dam there is a massive section capable of carrying the full load by its weight alone and, in addition, there is the resistance of the massive horizontal arch. For a less-than-gravity arch, regardless of the type, the load is carried primarily by a more or less highly stressed arch, with the certainty that if the arch should fail the vertical cantilever would also fail.

Stability of Arched Gravity Dams.—Many of the highest dams in the world are of the arched gravity type. Still higher dams of this type are under construction, and others even higher are proposed for the immediate future.

In one arched gravity dam investigated by the writer, no contraction joints had been provided. Contraction cracks on nearly vertical radial planes were formed, and a small amount of seepage was in evidence at the down-stream face. Water under reservoir pressure applied to the crack through a drill hole intersecting it at a depth of 10 ft. in from the surface, increased the seeped area, but the same pressure applied to the crack at 20 ft. from the face had no effect whatever upon the seepage.

In the great majority of arched gravity dams with which the writer is familiar, there never has been any indication of moisture at the vertical joints. There are others, not provided with joints, which have cracked on vertical radial planes, but which have never seeped at the cracks, or elsewhere. There are other arched gravity dams in which there are occasional spots in a vertical joint where seepage and even leakage appears, but these are rare and cover an insignificant fraction of the total number of joints.

The use of metal seals in the contraction joints near the up-stream face would not prevent the access of reservoir pressure to any open seams in the dam from which there is no leakage. It is the writer's observation that every void or seam in a masonry dam is filled with water under pressure. This was shown by the water which seeped out of the body of the St. Francis Dam for weeks after the failure. This seepage was apparently from the body of the concrete and was not confined to joints or seams.

It is interesting in this connection to note that dams in India, excellently constructed of rubble masonry laid in hydraulic lime mortar, are entirely free from cracks although they are straight in plan, but that the water seems to filter through them so freely that it oozes from the entire down-stream face which is completely covered with a film of water as if from a fine spray. This is only a temporary condition, however, and the dams soon become free from visible seepage.

An arched gravity dam of short radius is more stable than one of longer radius. For example, consider the extreme case in which the up-stream radius is equal to the thickness of the dam at a given elevation. The horizontal section would be triangular, and if the water pressure is to be confined to the up-stream face there will be no open joints and there will be arch action. If there are open joints, thus preventing arch action, the pressure at the up-stream face would be reduced by the back pressure on the planes of the joints, the summation of which would equal the area at the up-stream face, so that at the section at which the radius is equal to the thickness, if there are open joints, there would be no unbalanced water pressure at all to be resisted by the dam.

It follows from this that in an arched gravity dam assumed as being free from arch action by reason of open joints, as the radius of curvature is shortened, the stability is increased by the reduction of the effective water pressure.

In the special case of a dam without vertical joints, and in which the tensile strength of the concrete is greater than the contraction stress, there would be no cracks. In this case there would be tension in the arch and the effective pressure area would be that of the up-stream face. This would cause tension in the vertical cantilever, but it is reasonable to conclude that there would be the same resisting tensile strength in the concrete in a vertical direction as is assumed in a horizontal direction. This condition, moreover, would be avoided by the common practice of providing contraction joints at regular intervals.

Even if the existence of open joints is admitted, if the correct assumption of the effective pressure area is made, arch action can be neglected and an analysis which takes into consideration the reduction of section by convergence of the radii will show no tention in the up-stream face of an arched gravity section designed as a straight gravity dam with the lines of pressure at the middle-third points. Such an analysis will show that arch action is always present in an arched gravity dam, giving it a greater degree of security than from a straight gravity dam of the same section.

Uplift In and Under Dams.—Examination shows that the concrete in the St. Francis Dam was practically free from horizontal joints or seams. In a very few cases the concrete had been given a smooth surface on a horizontal joint and apparently had been allowed to set before new concrete was put on top of it. In such cases there seemed to be no union at all between the new concrete and the old.

On the other hand, concrete applied to a hard rock surface seems to become an integral part of the rock. The mica schist bed-rock adhered to the bottom of one of the overturned sections of the St. Francis Dam so firmly that the hard rock could be broken more easily than its union with the concrete. Drill cores from the bed-rock contact at many points of the Mulholland Dam showed a union of the concrete and the sandstone foundation apparently as strong as either the rock or the concrete. If, in placing concrete, care is taken to avoid "cold joints" by chipping away the surfaces which have set in air before placing the new concrete upon the set surfaces, it will be possible to avoid all horizontal joints both in the concrete and in its connection to a hard rock foundation.

The absence of stress due to uplift in the body of the concrete appears from the consideration that the water can act only in the voids. The pressure in a void in the concrete is in all directions, and, considering only the vertical pressures in a single void, it might appear that a tensile stress in the concrete would result; but considering that the upward and downward forces in each void are opposed and cancelled by equal downward and upward forces in the voids immediately above and below, it is readily seen that the summation of the vertical forces in the voids is zero, and that no vertical stress in the concrete can result from water pressure in the voids. This has also been shown by the application of enormous pressures to a liquid without effect upon the concrete or other porous substance immersed in it.

The uplift conditions appear to be the same at the junction with the bed-rock as in the body of the concrete, and it may be concluded that there is no uplift in the body of a homogeneous mass of concrete, or in a rock that is free from seams, or in a perfect contact between rock and concrete.

In the case of an open horizontal joint in the concrete or a horizontal seam in the rock, there is uplift for which allowance should be made in the designs of a straight dam, although in many, if not most, straight gravity dams, no such allowance has been made.

When the bed-rock is seamy and there are springs in the foundation, the possibility of uplift should never be ignored in a gravity type of dam, although certainly less provision need be made in an arched gravity than in a straight gravity dam.

Seepage from Dams.—Nearly all the high concrete dams built recently have been provided with an open gallery constructed in the concrete close to the foundation and to the up-stream face. Closely spaced wells drilled into the bed-rock discharge upward into this gallery and vertical drains paralleling the up-stream face of the dam discharge downward into it. There is a discharge from the wells and drains and usually also from seams in the concrete. The extent of this seepage varies with the character of the rock, and with the height and area of the water face exposed to reservoir pressure. There is less seepage from the softer shale or sandstone rocks and more from the harder granitic or dioritic rocks. The seepage from long high dams is much greater than from lesser dams and, as the rate of seepage is a function of the area of the dam face above the stream bed and also of the height, the ratio of seepage should vary with the three-halves power of the height.

Seepage water from dams usually carries lime in solution as calcium hydrate, which on exposure to air is converted into calcium carbonate. Calcium hydrate is always present in concrete, where it is formed by release from the cement in the reactions due to the process of setting. Seepage through new concrete usually ceases after a few months by the filling of its voids or by the sealing of its discharge by the deposit of calcium carbonate at or near the outer surface.

If water seeping through a seam in the concrete continues to deposit calcium carbonate without sealing the seam, there may be deterioration of the concrete and enlargement of the seam. All seepage of water carrying calcium hydrate in the galleries of a dam should be stopped by persistent caulking and pressure grouting.

The value of metal stops in contraction joints is open to question. Arched dams without such stops have proved to be perfectly free from seepage at the vertical joints. It is the writer's experience that there is never any trouble in preventing water from following the vertical construction or contraction joints in an arched dam. Even the most casual treatment of the vertical faces of such joints, by building one face against alternate shallow grooves and ridges formed in the other, is adequate; but it is extremely difficult to prevent seepage along horizontal "day's-work" joints if the previous layer has been allowed to set. It is necessary to provide the horizontal joints with many ridges or grooves in the concrete parallel to the axis, to clean the old surface thoroughly, and to prime it with mortar in advance of the concrete. With all these precautions there are occasionally seepages and unsightly stains at horizontal joints.

The utility and desirability of the vertical drains usually placed close to the up-stream face of concrete dams are questionable. They are supposed to reduce the percolation of water in the concrete and thus diminish uplift, and also to prevent the appearance of moisture at the down-stream face. Examination of several dams in which the vertical drains were omitted, shows them to be as dry as others under like conditions in which they were used. It would seem desirable to prevent the seepage of water through a dam by increasing the density of the concrete and the distance which the seeping water has to travel, and to avoid the use of any device which offers a short travel for the escape of water, either through the body of the concrete or through its seams or joints.

The use of sand cement is sometimes recommended as reducing the cost and decreasing the permeability of concrete, but it should never be used in a locality where frost action is possible. In one important dam, in a moderately cold winter climate, the use of sand cement has resulted in serious disintegration of the concrete wherever it is exposed to moisture and to winter temperature, although adjacent parts of the structure made of Portland cement concrete are entirely unaffected. The writer would always avoid the use of sand cement except possibly in a warm dry climate.

A very substantial economy in concrete for massive dam construction can be made by the use of large-sized gravel or cobbles. The cobbles are run through the mixer in measured proportions with the other aggregates. The first use of cobbles in concrete was at the Arrowrock Dam, where the maximum diameter was 5 in., but they have been used in gradually increasing sizes up to 8 in., or more, on the Don Pedro and Exchequer Dams. This use of cobbles in the proportion of about 25% of the volume reduces the cement without reducing the strength or the imperviousness of the concrete.

It has been shown that seepage at the down-stream face of contraction joints is usually caused by water flowing through open cracks or joints near the top where the dam is so thin as to be affected by temperature. For this reason, as well as for other conditions of design, it is desirable to make the top of the dam much thicker than stability requires, and a thickness of 30 to 40 ft. for very high dams is not too much.

Pressure Grouting of Contraction Joints.—The writer has considered herein only the case of dams in which no attempt has been made to increase the tightness of the joints by pressure grouting. In the writer's opinion pressure grouting would result only in the filling of the joints for a distance of 15 to 20 ft. inside the faces of the dam, and this would not appreciably increase the arch action in a massive arched gravity dam.

CONCLUSIONS

From the foregoing, the writer presents the following conclusions:

1.—Masonry dams on suitable bed-rock are safe and permanent structures and are most suitable for very high dams.

2.—An arched gravity type of masonry dam, whether built of concrete or of rock laid in mortar, has a larger factor of safety than a straight gravity dam of the same section.

3.—The contraction joints of an arched gravity dam do not prevent arch action.

4.—If the contraction joints should open so as to prevent arch action, water entering the joints would reduce, by its back pressure, the effective head on the dam and compensate for the loss of arch action.

5.—The stability of an arched gravity dam is increased as the radius of curvature is shortened.

6.—There is no uplift in the body of a concrete dam except that caused by horizontal joints, nor in foundation rock which is free from seams, nor in a perfect connection between the dam and the bed-rock, except that which may be due to a different degree of porosity in the concrete and the rock.

7.—Seepage through concrete may be detrimental and should be remedied by pressure-grouting, or by other means.

8.—The use of sand cement in cold climates should never be permitted.

CLASSIFICATION, SELECTION, AND ADAPTATION OF HIGH DAMS

By D. C. Henny,[3] M. Am. Soc. C. E.

Synopsis

In this paper the tendency to build dams of ever-increasing height is indicated. Foundation requirements for various types of dam are discussed; conditions limiting height are analyzed; and sources of danger are considered.

Historical Sketch

The progress in the design and construction of high earth dams during the Twentieth Century may be illustrated by reference to a report made in 1901 by an eminent Engineering Commission on changes in the New Croton Dam and the Jerome Park Reservoir of the New York Water Supply, which were then under construction.[4] The problem as to the New Croton Dam involved the safety of the earth dam portion as planned.

After investigating other earth dams, of the New York Water Supply, the Commission concluded that, with the material available, a dam higher than 70 ft. would not be safe. It recommended substituting solid masonry for earth in the high part of the dam, and that recommendation was approved by the authorities although it resulted in great additional cost.

This recommendation came as a surprise to engineers on the Pacific Coast, who were familiar with the Pilarcitos and San Andreas Dams, near San Francisco, Calif., and the Temescal and San Leandro Dams, near Oakland, Calif., all of which had been in successful service for twenty-five years at that time. Their respective heights are 95, 125, 115, and 125 ft. above stream bed.

The conclusion reached by the Commission was based principally on the fact that the water gradients in the other earth dams of the New York Water Supply System were found to intersect the down-stream slope high above the toe. The late Alphonse Fteley, Past-President, Am. Soc. C. E., then Chief Engineer of the Croton Aqueduct, questioned the correctness of the conclusion and, in the published discussions, drainage of the lower toe was mentioned as a means of lowering the plane of saturation.

Examples of High Dams

With a more complete understanding of the effect of such drainage and of selection or segregation of materials, engineers did not hesitate to build dams of ever-increasing height, so that the list of earth dams as high as 100 to 150 ft. has become a long one. The highest earth dam built to date is the Tieton Dam, in Washington, which is 233 ft. above stream bed. The Cobble Mountain Dam, in Massachusetts will be 245 ft. high.

---

[3] Cons. Engr., Portland, Ore.

[4] *Engineering News*, November 25, 1901.

The same evolution as to height is proceeding with all other types of dam. Among rock-fill dams may be mentioned the Morena Dam, in California (215 ft.), and especially the Dix River Dam, in Kentucky (275 ft. high), completed, respectively, in 1909 and 1925. The Pacific Gas and Electric Company is now (1928) building a rock-fill dam 300 ft. high on the Mokelumne River in California.

As to masonry dams of the gravity type, the straight Elephant Butte Dam, in New Mexico, is 240 ft., and the curved Arrowrock Dam, in Idaho, is 251 ft. above stream bed, with a height above the base of 308 and 349 ft., respectively. Both were completed in 1916 and held the record for some time, with the straight Kensico Dam, in New York (307 ft.), as a close third. These records have been broken by the Exchequer and Pardee Dams, in California, respectively, 330 and 350 ft. above the base; the former was completed in 1926 and the latter in 1929.

The Owyhee Dam, in Oregon, and the San Gabriel Dam, in California (the former of the arch gravity and the latter of the gravity arched type), will be 405 and about 490 ft. above the base, respectively. These heights, in turn, will be exceeded by the Boulder Canyon Dam, in Arizona-Nevada, which may reach a height above the base of 680 ft.

Examples of the single arch type of great height are the Shoshone Dam, in Wyoming, 320 ft. (1910), the Pacoima Dam, in California, 375 ft. (1928), and the Diablo Dam, in Washington, now under construction, to be 400 ft. above the base.

The same tendency toward greater height is shown in the buttress type of dam which includes the multiple-arch, Ambursen, round-head buttress, and dome forms of dams. The Lake Pleasant (multiple-arch) Dam, in Arizona, is 256 ft. high. The Rodriquez (Ambursen) Dam, in Lower California, Mexico, will be 225 ft. high. The Don Martin (round-head buttress) Dam, in Northern Mexico, a type suggested by F. A. Noetzli, M. Am. Soc. C. E., is 118 ft., and the Coolidge (dome) Dam, in Arizona, is 250 ft. high.

Among high dams on other continents are the Burring Juick (gravity) Dam, in Australia, 244 ft.; the Montejaque (arch) Dam, in Spain, 270 ft.; and the Tirso (multiple-arch) Dam, in Italy, 239 ft. above the base.

Questions arise as to whether the tendency to build dams of ever-increasing height will reach some definite limit beyond which it is not safe to go and what elements are likely to limit the heights of various types of dam.

DAM FOUNDATIONS

The foundation problem is at all times the most difficult, because in spite of elaborate geological study and underground exploration it is, in its nature, the most obscure; yet on it must mainly depend the selection of type and ultimate safety. Theoretically, it may be contended that a foundation is rarely so poor as to render the construction of some type of safe dam upon it impossible. Assuming the reservoir bed to have a sufficient degree of watertightness, defects of dam foundation can be overcome in general by a combination of type selection, deep penetration, and foundation treatment. Lack of hardness may dictate the adoption of a flexible type and a broad spreading of load and lack of tightness may compel the selection of a type with wide base, enforcing sufficient length of underground water travel. The complete rejection of a dam site is usually based, however, on economical considerations.

EARTH DAMS

Given a foundation free from such uncertain materials as quicksand and layers of marshy soil, and assuming the presence of the usual kinds of earth materials for construction—which, in themselves, or by segregation or mixing, can be made to supply earth of reasonable tightness for the up-stream part and material with free drainage qualities for the down-stream part—it is believed that no limit can be assigned to the safe height of earth dams, assuming the maximum height of water against them to be definitely controlled. Even in the case of some clay foundations which, when wet, may have small load resistance and when loaded may produce lateral pressures, safety can be obtained by proper proportioning of slopes and top width.

Lateral foundation movement may be absent in a low dam, but may take place in a high dam of the same proportions. Such movement took place in the Summer Lake Dam, in Eastern Oregon, a structure about 60 ft. high, of ordinary earth-dam proportions founded on clay. It settled, cracked, and forced the down-stream river bottom to rise about 3 ft. This was corrected by rebuilding to a flatter down-stream slope. An instance of settling occurred at the Lafayette Dam, near Oakland, Calif., which when it had reached 122 ft. of its ultimate 140 ft. of height above the stream bed, settled 20 ft. and produced a rise of ground under the lightly loaded toes. An engineering board has recommended less height and flatter slopes if this dam is reconstructed.

These instances illustrate the great difficulty of correct slope adjustment to wet clay foundation, but do not prove that a dam of great height cannot be built safely on such foundation.

ROCK-FILL DAMS

The rock-fill dams, which usually have steeper slopes than the earth dams, require foundations and building material of greater compressive strength. Furthermore, because of the high percentage of voids they require material which is free from weathering and water-softening so as to avoid excessive and, at times perhaps, sudden settlement and destructive bulging of the face slab. Any defect in this respect cannot be fully cured by the adoption of flatter slopes and, therefore, it imposes a limit on height. Greater height also may induce increase in volume of seepage and, therefore, requires greater resistance to erosion along areas through which the seepage may proceed. The greater base pressure of reservoir water also necessitates a higher degree of water-tightness of foundation material, the distance around the tight skin from full reservoir pressure to no pressure being far shorter than is available in an earth dam with broad contact.

Thus, it is held that local conditions impose a limit on the height of rock-fill dams to a degree far greater than is the case with earth dams. On the other hand, it is likewise held that with a hard and durable rock base, not subject to water-softening, with similar character of rock available for building and with a base water-tight, or capable of being made so by deep grouting, no reason exists for assigning a limit of height to this type of dam, short of that resulting from hardness of the building rock.

The combined earth and rock-fill type is a special adaptation of the earth dam. It may be superior to the common earth dam in cases where the latter has material of unsatisfactory drainage quality in its down-stream part, but requires a harder foundation to support the loose rock portion unless this is placed on a prepared foundation of the inverted filter type. It may also have advantages over the face-slab rock-fill type where broader contact for water-tightness is required with the foundation or where rock settlement is anticipated such as might endanger the effectiveness of a concrete face slab. In past examples of this type, as, for instance, the Twin Falls Dam, in Idaho, it has happened that percolation water has carried earth into the loose rock interstices, causing craters to form in the water slope accompanied by heavy loss of water. However, even in such cases, no serious danger to the dam has usually been involved, and it has been possible to stop flow by emergency measures. Such occurrences can be readily prevented by the interposition of a layer of crushed rock or gravel between earth and rock. It is believed that, with suitable foundation and building materials, dams of great height can be built in this type with fewer demands on tightness of foundation or hardness of rock than would be essential in the case of face-slab dams. All rock-fill types, however, as well as the earth dam, must be fully protected from overflow, as is illustrated by the failure of the first Lower Otay Dam.

MASONRY DAMS

The term, "masonry", is herein understood to include both stone masonry and concrete. For economic reasons concrete has almost completely displaced other classes of masonry in this country. The same tendency exists abroad.

Due to their rigidity masonry dams make far greater demands on foundation strength than the relatively flexible earth and rock-fill dams. In the latter, foundation loading changes gradually from the middle to the toes. In the former, it changes abruptly from maximum to zero.

In the gravity type of dam the load at the heel is almost solely governed by the height and the weight of the masonry and that at the toe, in addition, by the down-stream slope. The strength of the foundation rock is, therefore, likely to limit the safe height of a gravity dam within a far narrower range than is the case with a loose rock-fill dam. This, however, does not mean that dams of far greater height than those of the past cannot be built safely, but merely that requirements of hardness of foundation increase approximately in proportion to the height of the dam.

The same is true of the required strength of the building material which in past construction has probably never been stressed more than 600 or 700 lb. per sq. in. in compression. Concrete with a known strength of 3 000, 4 000 and even 5 000 lb. per sq. in. in 28 days is now readily secured, and there seems to be no reason why concrete in the base of a dam could not be safely subjected to compressive stresses of 1 000 lb. per sq. in., or even more (such as would obtain in a dam about 630 ft. in height), provided a source of satisfactory aggregate is at hand and the proper mix is used.

With increase of height, other requirements, such as water-tightness of foundation and of structure, rise correspondingly; so, also, do the demands on the drainage system under and in the dam. As far as the structure is concerned, these demands can probably be met by increased use of cement in the concrete and special care in bonding. Improvement of the foundation is least under control and the nature of the foundation is, therefore, the element most likely to fix the safe limit of height.

The buttress type of dam with its sloping face can be designed so as to tax the foundation to a lesser extent than necessarily results from the gravity type, despite the apparent concentration of load on the buttress bases. The resultant of dead and live load can be made to fall close to the center of gravity of the bearing area of the buttress, the proportions of which can be adjusted to local conditions, and maximum loads can be kept down, if desired, to little more than one-half those in pure gravity dams. The sliding factor can also be held down more readily than in gravity dams and the demands on foundation tightness are no greater, while the danger from uplift is much less. The dimensions of the web between the buttresses, whether arch, slab, or round-head, can be adjusted to the stresses. Thus, the strength and tightness of foundation rock place a closer limit on the height of the gravity than of the buttress type.

Arch action may be called to the aid of gravity dam support under a wide range of dimensions of dam sites. However, the single arch dam with nearly vertical axis is limited in its application to sites which permit the use of relatively short radii. With a given radius the arch pressure can be held below a desired limit, but secondary stresses are not so readily controlled. This is especially true of tension and compression stresses at the base. The introduction of steel where needed and of horizontal expansion joints at the up-stream toe forming a hinge, as was done by the writer in a dam constructed for the water supply of Ashland, Ore., or the adoption of the dome principle, as suggested and developed by C. R. Olberg, M. Am. Soc. C. E., and carried out in the Coolidge Dam, or a curved base support as was proposed years ago in the Ithaca Dam by Gardner S. Williams, M. Am. Soc. C. E., may in part solve this difficulty.

Arch dams make greater demands on the foundation in the abutments than the gravity dam, because a slight yielding under load has a profound effect on arch stresses. With a proper foundation the height of such dam will be largely controlled by the radius or radii to which it must be built.

Material progress has been made in the design of arch dams, recently aided by the results obtained from the Stevenson Creek Test Dam and tests with models, as sponsored by Engineering Foundation.[5] One valuable result has been the indication that deflections under various conditions of loading can be foretold with considerable accuracy by model tests. This will not only permit closer determination of stresses, but will also increase confidence in the correctness of design and in the safety of high arch dams.

---

[5] *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3.

FLOOD DANGER

Dams blocking the discharge from large drainage areas are subject to potential danger and damage from flood overflow during construction.

It is often possible to complete a low earth or rock-fill dam above the base, or that portion of a higher dam left open for safe flood passage, within a season between floods. When this cannot be done and sufficient by-pass capacity cannot be feasibly provided, risk from overflow during construction must be taken, and this risk becomes greater with increasing height and volume of dams. If this risk cannot be assumed, seasonal measures must be taken for permitting safe overflow as, for instance, through a temporary flume over the down-stream slope of the dam, as was provided in the building of the hydraulic-fill dam near Conconully, Wash., or by paving the down-stream section with concrete as was done in the Nexaca Dam, in Mexico.

The consideration of floods may impose a severe restriction on the height of earth and rock-fill dams. The difficulty of estimating maximum floods is well understood as is also the danger of under-estimates which have been the cause of several major disasters in the past. Presumably with these considerations in view, Italy, by governmental regulation, has limited the height of earth dams to 20 m. (66 ft.) and of rock-fill dams to 25 m. (82 ft.).

It is probable that some earth dams standing in the path of flood discharge from large drainage areas and now deemed safe because of assumed sufficiency of spillway capacity, are in actual danger from flood overflow and, in places where great height and especially large storage are involved, public interest may demand immediate attention.

In the case of masonry dams danger from overflow during construction, even when no apron has been provided, is usually not great. The uncompleted structure has excessive strength to resist water load and would usually be stable even after a serious undermining of the toe due to overflow erosion. Attack on the foundation is generally most serious along the abutment toes where a lateral current may be set up. Therefore, it seems wise to confine emergency overflow to the central part of the dam unless special apron protection is provided along the slopes.

The arch form of dam renders the safety of the structure less dependent on the preservation of rock integrity below the toe than is the case with the straight gravity dam. Uncertainty as to maximum flood flow may be in some cases a sufficient argument for incurring additional expense, which may be involved in curving gravity dams.

EARTHQUAKE AND EARTH MOVEMENT DANGER

The danger of earthquake and earth movement is a serious factor in the selection of type and, in fact, in deciding whether any type of high dam can be safely built, especially on the Pacific Coast. The scrupulous search for a possible fault crossing the dam site and the geologic estimation of the occurrence of last movement along such fault, therefore, have become common practice.

Very severe earthquake shocks may be borne safely by straight gravity dams and earth dams, as well as by pure arch dams. This was well illustrated by the undamaged condition of the straight concrete Crystal Springs Dam and the earth San Andreas Dam after the 1906 earthquake in San Francisco and of the pure arch Gibraltar Dam after the Santa Barbara earthquake in 1925. Since the gravity dam depends on its dead weight for safety and not on the continued integrity of the entire structure, as is the case with an arch, it is probably more secure from earthquake damage. The earth dam is liable to be overtopped by a tidal wave and, on the whole, the gravity dam must be regarded as the safest, especially if curvature in plan gives it additional safety against sliding and if its cross-section has been determined on the basis of converging radii and of possible uplift. These considerations led to the selection of the arched gravity type instead of the more economical arch for the East Park Dam in Northern California completed after the San Francisco earthquake.

If a dam must be located across a fault line the gravity type has distinct advantages, because even if sheared, its parts would probably otherwise remain in place and oppose their mass to a rapid release of stored water. The semi-flexible Ambursen dam with buttresses straddling the fault may also be regarded as safe, at least against minor earth movement along the fault line, and this type was selected in preference to the multiple arch for the 120-ft. Stony Gorge Dam, in Northern California, which crosses a fault.

The ordinary earth dam is unsafe if it crosses a fault line along which movement may take place. This was evidenced after the San Francisco earthquake in 1906 by the condition of the first or Upper Crystal Springs Dam, which crosses the San Andreas fault. The body of the dam was found to be badly fissured, both laterally and longitudinally, and it would have been destroyed had not the water stood at the same height on both sides of the dam, owing to the backing up of reservoir water by the Crystal Springs (concrete) Dam below.

CRACKING OF CONCRETE

Another and possibly serious danger in cases where mass concrete is used in the construction of high masonry dams (which is now usual in the United States for economic reasons) is the tendency of concrete to crack after dissipation of internal heat.

When concrete sets, its temperature is raised and it will expand until compressive strength develops and expansion meets with resistance. Further heating will set up compression. Subsequent cooling will reduce compression until it is completely neutralized, after which further cooling tends to produce tensile stresses. Increase of load will tend to reduce these stresses, whereas drying out of the concrete will tend to increase them. The existence of shrinkage cracks in many concrete dams shows that the tensile stresses exceeded the tensile strength of the concrete. This is not surprising when it is considered that a heating of concrete to 30° Fahr. and more is not uncommon, and that a cooling of 15° is likely to produce a tension of more than 150 lb. per sq. in.

Cracks will naturally show first on the sides of large dam blocks having the longest horizontal dimension and where air cooling and possibly drying have induced tension. These cracks may penetrate only a short distance into the concrete and may tend to close, after the lapse of time, as the interior mass cools. With further interior cooling the outer concrete that previously cracked under tension, by interior pull, may be placed under compression, and this interior pull may become intensified until previous skin cracks extend partly or wholly through the mass.

In concrete dams, the thickness or base length of buttresses increases with height, and the tendency to crack becomes aggravated by the greater internal heat generation due to higher cement content and relatively smaller radiating surface. In gravity dams, the tendency to crack in a transverse direction has usually been counteracted by the provision of contraction joints even if transverse shrinkage cracks approximately normal to the axis of the dam contain no element of danger. Cracks parallel to the axis of the dam, whether in the mass of a gravity dam, or in the buttresses of a buttress dam, may be of serious consequence, especially as such cracks are likely to be approximately vertical and are thus often close to the direction of maximum shear. The presence of such cracks may well become equivalent to a dangerous narrowing of the base. In buttresses, it is generally feasible to introduce sufficient temperature steel to prevent them, as was done, for instance, in the case of the Stony Gorge (Ambursen) Dam, in California, where, in the early stage of construction, such cracks became evident. In gravity dams, prevention of shrinkage cracks by the use of steel would be enormously expensive.

In the buttresses of the Coolidge Dam and the 165-ft. Big Dalton (multiple-arch) Dam near Los Angeles, Calif. (both completed in 1929), contraction joints parallel to the axis of the dam and to the direction of the principal stresses have been introduced. The same provision has been made in the design of the San Gabriel (arched gravity) Dam now under construction near Los Angeles.

In addition, the forming of shrinkage cracks may be counteracted partly by a suitable construction progress program. Such a program may prohibit over-rapid construction and may require each concrete layer to remain exposed with the surface kept moist for a specified length of time to effect maximum cooling. It may reduce the height of rise from 5 or 4½ ft., which has become usual, to 4 ft., or even less. Under extreme conditions it may call for pre-cooling of the mixing water and aggregates. It may also specify that each step of the work be brought up level or nearly level from end to end. In this way expansion of fresh concrete, as it heats and acquires compressive strength, will meet with early resistance from adjoining concrete, and subsequent cooling must first cause compression to be neutralized before tension can develop.

UPLIFT

The possible dangerous effect of uplift either at or below the base of gravity dams or higher up in the concrete mass has been long known. It may re-distribute loading in an injurious way and in all cases it increases the tendency to slide.

In modern dam design provisions are made to reduce or prevent uplift, such as a deep cut-off wall with or without a grouting curtain and a system of drainage. In addition, allowance for uplift is generally made in the design, but great differences of opinion exist, not merely as to the pressure gradient to be assumed (in regard to which valuable experimental data are being accumulated), but especially as to the proportionate area over which this pressure may be considered effective. Attempts at experimental determination undertaken recently at the Bull Run Dam have yielded negative results because water-creep through construction joints in mass concrete seems to be due to unavoidable imperfections of actual work which cannot be duplicated in the laboratory. Generous allowance for uplift in the design, strenuous efforts to obtain water-tightness and perfect bond, together with ample and effective provisions for drainage, are the only safeguards which have been developed thus far independent of measures to increase the resistance to sliding, such as enlarged thickness and curving the dam in plan.

The design manifestly should aim at opposing the movement of water through the up-stream part of the dam and facilitating it after this barrier has been once passed. From this point of view keys in contraction and construction joints in the down-stream part of the dam are objectionable.

Construction joints are planes of weakness both for sliding and for penetration of water which may exert uplift. It is an advantage, therefore, to avoid the usual horizontal joints and to require joints to curve upward in a down-stream direction, so as to become normal to the dry slope as nearly as ordinary concrete deposition will permit. In this manner, the incidental advantage of reduction of maximum shear in construction joints will be secured.

Tension in Foundation

Finally, attention must be called in the case of masonry and, especially gravity, dams to the possibility of tension in the foundation rock near the heel. This matter was ably discussed[6] by Messrs. Ottley and Brightmore in 1908. The subject was treated by them both theoretically and experimentally. The tension under a full head of 30 in. in the model was calculated and for a shape of ordinary proportions was found to be about 1.2 lb. per sq. in., which for a height of 400 ft. would be equivalent to about 200 lb. per sq. in. The direction of maximum tension was also calculated and the existence of such tension and its computed direction were confirmed by cracks in the plasticine models.

The serious feature of such tension lies in the fact that it increases with the height of the dam; that the direction of possible cracks or the widening of existing rock fissures is such as to extend from the heel downward under the dam; and that the opening thus formed may cross the assumed vertical zone of foundation tightness, whether natural or procured by grouting. By such opening direct communication may be established between open reservoir and the foundation drainage system which may be overtaxed thereby, with the possible result of increased uplift and erosive seepage.

---

[6] *Minutes of Proceedings*, Inst. C. E., Paper W-3674, Vol. CLXXII, 1907–08, Pt. II, p. 89.

It is true that the effect of such cracks has not been observed in actual dams, but nevertheless cracks may exist and they may become more pronounced in dams of greater height. The danger may be counteracted to some extent by spreading the base of the dam in an up-stream direction and introducing steel as was done in the case of the Bull Run Dam, even if this dam is not of unprecedented height and the base rock is strong and remarkably free from fissures. The same measures are specified for the San Gabriel Dam with the added precaution (understood to be common in Germany) of placing a triangular body of plastic clay along the up-stream toe.

CONCLUSION

A heavy responsibility rests on the shoulders of engineers engaged in the design and construction of dams of ever-increasing height. Information which may assist in avoiding danger is being accumulated in many directions.

HIGH DAMS: THE VIEWPOINT OF THE GEOLOGIST

By F. L. Ransome,[7] Esq.

Introduction

If rocks were entirely homogeneous it would probably rarely, if ever, be necessary to call upon a geologist to examine a site for a high dam. It could safely be assumed by the engineers that the rock exposed at the site continues unchanged in horizontal and downward directions, and that consequently the appropriate tests for strength, imperviousness, and other properties, applied to samples from the site, would determine whether a dam could safely be constructed at that locality. At first thought, the necessity for considering the effect of earthquakes might be regarded as an exception to the foregoing general statement, but serious earthquakes are a consequence of movement along faults, and faults are obviously departures from homogeneity.

As a matter of fact, rocks are never perfectly homogeneous for any considerable distance. The nearest approach to the ideal is probably to be found in large intrusive masses of rock that have solidified from a molten condition, at great depth, and have been exposed at the surface only after long erosion. These are the plutonic rocks, which are of interest to the geologist, and of which granite is the most familiar example. Even in granite, however, it is necessary to reckon with the possible existence of faults; with various types of fractures; with zones of crushing or shearing; with included masses of other rock, such as schist; with later dikes of different kinds; and with contacts between the granite and other rocks.

In layered rocks of many sorts, the lack of homogeneity is more obvious. A compact lava flow, of basalt for example, may be spongy or vesicular in its upper and lower parts, or may rest on a layer of conglomerate or of loose lapilli. A bed of hard sandstone may be succeeded by a bed of soft, soluble gypsum, or even of rock salt. A stratum of limestone may be overlain by a layer of shale. An upturned bed of quartzite may be separated by a belt of slate from a belt of mica schist, etc., in almost infinite variety. These relations, moreover, may be further complicated by igneous intrusions and by faulting.

It is in the recognition and interpretation of such departures from homogeneity that the chief usefulness of the geologists in relation to dam sites lies. Such essential elements in the geological structure may not be apparent at the actual site and yet may constitute vital factors in its proper selection. To recognize them and to grasp their significance in relation to a proposed dam is likely to require a knowledge of the region such as can be acquired only by geological methods of investigation and by geological experience.

Clearly, the fundamental requisites in the rocks at a site for a high dam are: (1) That the rocks be strong enough to withstand the stresses transmitted from the structure; (2) that they be reasonably impervious; (3) that they neither change in volume, nor soften, nor dissolve, under the action of water; and (4) that they be not subject to such movement as would damage or destroy the structure. It is proposed, in what follows, to consider the relation of various rocks and geological structures to these fundamental requisites.

---

[7] Prof. of Economic Geology, California Inst. of Technology, Pasadena, Calif.

Geological Requirements for a Reservoir

A geological investigation in connection with a proposed dam site commonly comprises two parts—that of the reservoir as a whole and that of the actual dam site.

Failure of a reservoir, where the dam remains intact, is rarely catastrophic. It may, however, mean the loss of valuable water, or such large leakage that the reservoir becomes wholly useless. Hardness or strength is seldom an important consideration in the rocks of the reservoir as a whole. Even porous rocks, provided that the open spaces are not connected so as to form continuous channels, need cause no concern. Freely permeable rocks and soluble rocks, however, require particular attention and must be considered in relation to the geological structure of the region. Examples of freely permeable rocks are beds of gravel or poorly cemented conglomerate and some volcanic tuffs or breccias. Basaltic lava flows may be so frothy or cavernous, particularly near the top and bottom of each flow, as to permit the passage of large volumes of water. Ordinary compact limestone, although its solubility under normal conditions and for such periods as need be considered in engineering practice is usually negligible, is nevertheless geologically soluble, as is plainly shown by the occurrence of such great caverns as those near Carlsbad, N. Mex., and the familiar underground streams of some limestone regions. Consequently, in places where a proposed reservoir is to be partly or wholly on limestone, it becomes necessary to determine, if possible, whether there are any existing caverns or subterranean chambers through which the water might escape.

Wherever freely permeable rocks occur in a reservoir site a knowledge of geological structure becomes highly important. Clearly, for example, a permeable bed which outcrops in the reservoir and, by maintaining a regular dip, appears again along the banks of a stream a few miles distant at a lower elevation, would conduct water away from the proposed reservoir, whereas, if the permeable bed is dislocated by a fault, or folded so that there is no hydraulic gradient to any lower outcrop, it might not permit any serious leakage.

Where permeable rocks are present, the position and slopes of the underground water surface, or water-table, should be ascertained. In most drainage basins, the water-table practically coincides with the topographic surface near the streams and rises toward the ridge crests, although at a more gradual slope than the surface of the ground. Such a condition indicates that the underground water of the basin is moving slowly toward the streams that drain the basin, and, consequently, that the proposed reservoir will be more likely to receive than to lose water by underground flow. On the other hand, if the water-table in some part of the basin is found to be lower than at the adjacent streamways, this indicates that some water is escaping from the basin through underground channels and, consequently, that still more might escape were the basin converted into a reservoir. Obviously, in a reservoir formed merely for flood control, such loss would be immaterial and might even be advantageous.

Among the soluble rocks most likely to require attention in reservoir projects are gypsum and rock salt. The McMillan Reservoir, near Carlsbad, is partly a failure because the water escapes in large quantity through underground channels in gypsum and these are increasing in size. On the other hand, Willow Lake, another reservoir about 20 miles south of Carlsbad, is enclosed by gypsum, but has shown no leakage since the dam was built, about 1922. The difference is probably to be explained by the supposition that, in the case of the McMillan Reservoir, there was initially a small natural solution channel through the gypsum, which the water enlarged rapidly by further solution, whereas, at Willow Lake, no such passage exists and, although gypsum is going into solution everywhere around its shore, this general solution has not led to any leakage. Rock salt is, of course, even less trustworthy than gypsum as a water barrier. Thick beds of salt occur in the proposed Boulder Canyon Reservoir, along the Virgin River, Nevada, but their position is such that they cannot cause leakage. The question that has been raised as to whether these salt beds would render the water of the proposed reservoir too saline for use, is a reasonable one, and indicates a possibility that should not be overlooked. In this particular case the effect of solution would be to blanket the salt with silt from overlying beds and the quantity of salt dissolved would probably be negligible in so large a body of changing water.

Significance of Topography at a Dam Site

As the purpose of most high dams is to impound as much water as is practicable behind a structure of minimum size and cost, the site is usually a narrow canyon above which the stream flows through a valley or basin. The existence of a steep-walled, narrow canyon generally indicates that the rock at the canyon is harder than that in the up-stream valley. This indication, and the fact that canyons are geologically youthful features, determine the favorable condition that the rocks in canyon walls are as a rule well exposed and relatively fresh, particularly near and under the stream. It is often comparatively easy, therefore, to ascertain the character and structure of the rocks in the canyon and to recognize transverse faults, if they are present. These faults are the generally favorable features of a canyon location.

It is advisable, however, to determine why a stream forsakes an apparently easier course (as is frequently the case) and turns aside to cut a canyon through hard rock. It is particularly important to ascertain whether this apparent preference is due to a fault or other particular line of weakness along the canyon and concealed by the sand, gravel, and boulders of the present stream bed. This may be a complex geological problem requiring an understanding of the development of the topography of the region. The river may have had its course determined by rocks and structures that have been completely removed by erosion and, having once cut into the formerly buried, harder rock, has been forced to remain within it. In this case, there would be no reason to expect the existence of a line of weakness under the stream bed. On the other hand, some smaller stream, with more direct path to the sea and steeper gradient than the main river, may have captured the larger stream by head-water erosion. If so, there may be a belt of weak rock under the bottom of the canyon. As a rule, the courses of faults are more nearly rectilinear than the courses of streams. Consequently, the presence or absence of a fault along the bottom of a canyon can usually be determined by an examination of spurs and ridges that extend across the prolongation of a line drawn through the part of the canyon at the dam site. For example, at the site of the Owyhee Dam in Eastern Oregon, the drill records suggested the presence of a fault under the river. Examination of the ground showed a little ravine and saddle, in line with the gorge, and the suggestion was made that a trench be dug across this ravine with a view to exposing the fault zone. The fault was found, as expected, and provisions to deal with it were embodied in the design of the dam.

THE ELEMENT OF INITIAL STRENGTH IN THE ROCKS AT THE DAM SITE

It may fairly be assumed that the rocks at a dam site should be at least as strong as the material of which the dam is to be built. Ordinarily, this means at least as strong as good concrete. The actual strength of sound samples of the rock can readily be determined by the usual engineering tests. It is for the geologist, however, to point out the bearing of geological structure on the strength of the rock mass as a whole. For example, a site may be in granite which shows a crushing strength of 25 000 lb. per sq. in. and yet the presence of zones of shattering or shearing may make the site unsafe for a high dam.

In stratified rocks, the attitude of the beds must be taken into consideration. As every quarryman and mason knows, stratified rocks are stronger in directions perpendicular to the bedding planes. Ordinarily, an up-stream dip of the beds would appear to be more favorable to dam construction than a down-stream dip. Yet, with beds of different degrees of permeability, conditions might exist that would make a down-stream dip preferable. In beds that are nearly horizontal, or in steeply inclined beds, striking approximately parallel with the stream, the possibility of the slipping of one bed past another can not be ignored.

This last consideration applies particularly to shales and to slates and schists. The St. Francis Dam is believed to have failed first at its northwestern abutment, but the southeastern abutment was of little better quality. The dam at that end was built against schist in which the thin laminations stood nearly parallel with the slope of the canyon wall. The concrete was poured against the smooth cleavage faces of the schist and adhered to them, but there was little more adherence between the schist laminations themselves than between the sheets of a pile of paper.

Joints in rocks have a bearing on the strength of the mass, but they are also important with reference to permeability and will be considered more fully under that subject.

Without going further into detail, it is sufficient to emphasize the fact that, as regards the initial strength of the rocks at a dam site, it appears to be the particular function of the geologist to call the attention of the engineers to the effect of structural features which may not be obvious to them, but which can be recognized by those who have had special training in geological observation and interpretation.

THE ELEMENT OF PERMEABILITY IN THE ROCKS OF A DAM SITE

In considering the behavior of rocks with respect to leakage around a dam, it is necessary to distinguish between porous and permeable rocks. Certain volcanic rocks, such as vesicular basalt, or more or less pumiceous rhyolite, may be eminently porous, but yet, because of the lack of connection between individual pores or cavities, these rocks may be relatively impervious.

As it is the essential function of a dam to retain water, leakage is to be avoided where possible. Nevertheless, there is a wide range between permissible or controllable leakage and conditions under which the escaping water is a serious loss or endangers the entire structure. The slow seepage of water through a porous but insoluble rock, such as a spongy lava, may cause no important loss from the reservoir; it is more likely to decrease than to increase, and can do no harm. On the other hand, even a small leakage through rocks that are partly soluble may enlarge the original openings, or, by removing the cementing constituents, so weaken the foundations of the dam as to cause disaster.

When a dam is built on stratified rocks it is important to know the stratigraphic section in detail and to consider each different kind of rock with reference to possible leakage and the effect of such leakage.

Fractures in the rocks may be classed as faults and joints. A fault is a fracture of considerable horizontal and vertical persistency, along which the rock on one side has moved past the rock on the other. The displacement may be a few inches or several miles. A characteristic feature of a fault is a seam of clay-like material, known as gouge, which has been produced by the grinding together of the rock walls during the movement on the fault. The seam of gouge may range from a fraction of an inch to 8 ft., or more, in thickness. It is ordinarily accompanied on one side, or on both sides, by a zone of shattered or crushed rock. The gouge is usually impervious, but the shattered rock is likely to be freely permeable. Consequently, faults, where present, must be considered as probable paths of leakage. A fault, it is important to note, cannot be removed by excavation. If nearly vertical, it may be expected to continue far below the bottom of the dam and, if nearly horizontal, far beyond the dam, in all directions.

Joints are relatively short fractures along which there has been no appreciable movement of one side of the crack past the other. Consequently, they contain no gouge. They are present in most rocks, and, when long exposed to the weather, as in canyon walls, they may become superficially accentuated and give such an alarming appearance of general fragility to the rock mass as has led some engineers and geologists to an unjustified condemnation of particular sites.

The fine-grained granitic rocks of Boulder Canyon, on the Colorado River, are dissected in many places by innumerable joints. Yet short tunnels driven into some of the most thoroughly jointed rock have shown that within 10 to 20 ft. from the surface the joints become scarcely visible and the blocks into which the granite is divided are so irregular in form and so tightly fitted together that no important leakage could take place. As granite is practically insoluble and unaffected by water, no material could be removed, and, in a practical sense, the presence of the joints is negligible. Moreover, joints in hard, insoluble rock, if they are open enough to cause serious leakage, are amenable to grouting.

THE EFFECTS OF SOLUBILITY IN THE ROCKS AT A DAM SITE

Solubility is, of course, a relative term. Probably no rock is completely insoluble under all conditions found in Nature. The solution of rocks like granite, quartzite, and basalt, however, is so slow or so slight as to be practically negligible. On the other hand, rock-salt and gypsum are dissolved so rapidly in flowing water as to make them utterly unfit for the foundation of a high dam. Limestone and marble occupy an intermediate position between these two groups. That limestone is geologically soluble, especially in water containing carbon dioxide, has already been pointed out. Yet, with proper precautions against leakage, it would be practically safe to build a high dam on compact limestone.

The fact that calcite or calcium carbonate is the most common cementing material in ordinary sandstones and conglomerates lends particular importance to the question of the solubility of this substance. Many such sandstones and conglomerates are fairly resistant to weathering and stand out from the surface in bold outcrops. Nevertheless, the questions may be asked—Are such rocks, with a calcareous cement, suitable for the foundation of a high dam? Will water, seeping through the rock under pressure, dissolve enough of the calcium carbonate to weaken the rock seriously or to destroy its coherence? Quantitative data to answer these questions are apparently lacking, and this deficiency offers an opportunity for some experimental engineering research on the subject. In the meantime, the opinion may be offered that with good contact between the concrete of a high dam and a reasonably hard and well-cemented sandstone or conglomerate, there need be no apprehension that enough of the calcite binding material will be removed to weaken the rock appreciably, especially as reservoir waters, as a rule, do not contain much carbon dioxide in solution. In the presence of carbon dioxide, calcium carbonate goes into solution as calcium bi-carbonate, but is deposited again as carbonate when the carbon dioxide escapes or is driven off, as in the familiar example of the incrustation in the bottom of a tea-kettle. Calcium carbonate, therefore, although taken into solution at one place in the rock may be deposited in another place, close by. In this respect it differs from gypsum, which is carried away in a more persistent state of solution.

The Action of Water on Concrete Dams.—It apparently is not uncommon to find that water, passing through the concrete, deposits in inspection galleries or on the under side of the concrete slabs of dams of the Ambursen type, an incrustation that consists chiefly of calcium carbonate. The question has been raised: Does this calcium carbonate represent merely free and unessential lime, leached from the concrete, or does it indicate a loss of some constituents to which the cement owes its binding power, and, consequently, a deterioration in the strength of the concrete? As far as the writer has yet been able to learn, there has been no thorough investigation of this important question. There appears to be an opportunity, therefore, for co-operative research on the subject on the part of the cement manufacturers, the National engineering organizations, and the U. S. Bureau of Standards.

It is suggested that it would be advisable, also, to investigate the solvent action of water on clastic rocks—that is, rocks composed of fragments held together by some natural cementing material. With findings based on such an investigation a standard method of testing such rocks might be devised, with a view to predicting their behavior under the action of water at various pressures.

In some localities gypsum occurs as a cementing material in sandstone. If there is any possibility of the presence of the mineral in this rôle at a proposed dam site, the rock should be carefully tested. A gypsiferous sandstone can not be regarded as a safe foundation for a dam.

Softening or Disintegration of Rocks in Water

Some clastic rocks, particularly those held together by films of clay, may be fairly hard when dry, but become soft and plastic when wet. This change is sometimes so pronounced as to be rather startling, even to an experienced geologist. The quantity of clay requisite to produce this effect is not necessarily large. It may be a mere film at the contacts of the constituent rock particles.

Probably the most tragic and impressive illustration of the importance of recognizing this property when present in the rocks of a dam site was furnished by the failure of the St. Francis Dam, in 1928. The northwest abutment was on conglomerate, belonging probably to the Sespe formation, of supposedly Oligocene age. The conglomerate, probably a fresh-water deposit, is composed of poorly assorted schist, granite, and sandstone detritus, which ranges from clayey silt to pebbles 8 in., or more, in length. The bedding is generally rather indistinct and much of the formation is covered with soil.

When dry the conglomerate is fairly firm, although it is by no means a strong rock. A test gave a crushing strength of 500 lb. per sq. in., whereas the concrete of the dam has a strength of 2 000 lb. per sq. in. When wet, however, the rock shows a remarkable change. A piece of the conglomerate, when placed in a beaker of water, begins to flake and crumble and soon slumps to the bottom of the vessel as a loose gritty sediment that can be stirred about with the finger. Evidently, the particles of the conglomerate are held together merely by the films of clay and when the mass is wet all cohesion is lost. Even the fractured and faulted pebbles fall apart into small fragments.

The conglomerate is also traversed by many small veinlets of gypsum and it is probable that this mineral constitutes a part of the cement that holds the rock together when dry. The solubility of gypsum is well known, and the presence of this mineral probably contributes to the rapid disintegration of the mass when wet.

Obviously, a simple immersion test would have revealed the utter unsuitability of this material for the foundation of a high dam.

Other rocks, particularly some soft sandstones, show a decrease in strength when wet, although the change is not often so pronounced as in the conglomerate described. For example, specimens of the soft, massive sandstone from the Glenn Canyon site, near Lees Ferry, Ariz., show a decided softening when soaked in water.

It is the part of prudence to subject any clastic rock to an immersion test before accepting it as a safe foundation for a dam. Care should be taken, however, to obtain a fair sample and to avoid any previous loosening of the material by shock, such as might easily result from blasting.

Some fault gouges, although soft and plastic when wet, are fairly hard when dry. It is necessary to keep this point in mind when conclusions as to the possibility of recent movement along a fault are based on the character of the gouge.

CHANGES IN VOLUME OF ROCKS

The possibility of a change in volume in the rocks at a dam site rarely requires practical consideration. Clays, as is well known, shrink or swell with variations in the water content and anhydrous calcium sulfate or anhydrite increases in volume when it takes up water to form gypsum. These materials, however, are to be avoided in a site for a high concrete dam, on other grounds than that of possible volume changes.

Serpentine results from a slow change, involving hydration, from basic igneous rocks of the peridotite group. This alteration is accompanied by change of volume and by movement within the mass recorded by irregular shearing and smooth curved surfaces of slipping. The general unsuitability of such rock for the foundation of a high dam is due rather to the consequences of an alteration that has already taken place than to the possibility that serpentinization is a process that is sufficiently rapid or active to produce changes in the rock during the existence of the dam.

STABILITY AS AFFECTED BY FAULT MOVEMENTS AND EARTHQUAKES

A fault, once initiated, is likely to continue to be a plane of weakness and may show recurrent movements separated by long periods of quiescence. For example, there is strong evidence for the conclusion that the great Verde Fault in the Jerome District, Arizona, after a throw of 2 400 ft. in pre-Cambrian time, remained immobile throughout Paleozoic and Mesozoic time, but was active in late Tertiary or Quaternary time, with an additional throw of 1 600 ft., or a total of 4 000 ft. Renewal of movement, after long periods of rest, also took place in some faults in the Grand Canyon region of Arizona.[8]

In some faults, as may be observed in Boulder and Black Canyons, on the Colorado, the gouge has become lithified and is now a fairly hard rock. It appears safe to conclude that movement on such faults has not taken place for hundreds or thousands of years. Confirmation of this conclusion, as regards the faults in Black Canyon, was found in the fact that at one place on the brink, about 600 ft. above the river, smooth, water-worn channels were found in the rock, accompanied by pot-holes and river boulders. These channels could have been worn only at a time prior to the cutting of the present gorge. As one of the channels passes smoothly over one of the faults, it is clear that the fault has not moved since the river began to cut its present gorge.

---

[8] See *Science*, Vol. 27, pp. 667-669, 1908.

In faults with soft clay gorges, it is usually difficult to estimate the probability of renewed movement in the future. An examination of historical records and of the topography may give some clue as to the recency of movements in the past and may supply some basis for a prediction as to the future. Faults with soft gouges, however, are uncertain features and should be avoided at a dam site, where this is practicable. The mere presence of a fault does not justify the condemnation of a site. It is important, however, that its existence be known, that the probability of future movement be carefully considered, and that the dam be designed to meet future contingencies, so far as this may be possible. Of course, no dam should be built across a fault known to have been active in recent times.

Mere proximity to a known zone of active faulting does not necessarily imply serious danger to a high dam. Such dams are usually of concrete and are founded on solid rock. It is re-assuring to remember that the Crystal Springs (concrete) Dam, near San Mateo, Calif., was not injured by movement on the San Andreas fault in 1906, although the fault line passes only about 900 ft. west of the dam.

Major earthquakes are usually a consequence of fault movement and the two phenomena must be considered together in relation to their effects on dams. Earthquake waves have been divided into two general classes: (1) Condensational and distortional waves, propagated with high velocity, as a consequence of the elastic property of rock; and (2), waves visible at the surface of the ground as slowly traveling undulations.

Waves of the first group rarely exceed an amplitude of 0.5 mm., although in one Japanese earthquake an amplitude of 76 mm. (about 3 in.) is recorded.[9] The origin of the slow visible undulations of the second class is not fully understood. Such waves may be 0.3 m., or more, in height and 10 m. from crest to crest. They are apparently produced by the passage of the elastic waves from rock into loose, unconsolidated material, such as alluvium. There is no satisfactory evidence that these large waves traverse solid rock and the nature of this material would seem to preclude such an occurrence.

In considering the possible effect of an earthquake on a dam, it is well to remember that a properly constructed high dam is built on solid rock and becomes in fact an integral part of the rock upon which it rests. Such a structure probably would be affected only by the short elastic waves that are propagated with high velocity through rock and not by the visible secondary waves that pass as terrifying and destructive undulations through alluvium or artificially filled ground. It is permissible, therefore, in a region subject to earthquakes, to examine the natural rock forms and to deduce from their condition the probable effect of future earthquakes on a concrete dam. This reasoning was applied to the proposed dam site at Angostura de Teras, Sonora,

---

[9] "A Manual of Seismology," by C. Davison, Cambridge, 1921, p. 38.

Mexico. This region is known to have experienced a severe earthquake in 1887, in which some lives were lost and damage was done to buildings in at least two small towns or villages.

In comparison with other regions of similar topography and climate, but not known to be subject to earthquakes, the rocks in the vicinity of Angostura de Teras are not exceptionally broken or shattered, and the cliffs show no more than the normal quantity of rock débris at their bases. Particularly significant in such a comparison is the Angostura de Vallecitos, a very narrow gorge in rhyolite, a few kilometers east of the dam site. The walls of this gorge are practically vertical; in places they overhang, and are at least 70 m. high. It would appear that any shock capable of destroying or seriously injuring a concrete dam would have shaken down these walls. An even more reassuring exhibit in the vicinity of the site is a slender residual spire of conglomerate, about 35 m. high, which obviously passed unscathed through the earthquake of 1887.

In a region where such relatively instable rock forms as those just cited have survived the only severe earthquake recorded in historical times, it is reasonable to conclude that a well-constructed concrete dam, founded on solid rock, and situated in a canyon, would not be damaged by a future shock of equal intensity.

In conclusion, it may be remarked that dam sites which, from the geological point of view, are ideal in all respects are exceedingly rare, if any exist. On the other hand, it is a tribute (perhaps not unduly optimistic), to the skill of modern engineers to state that a dam may be built successfully on any site, if the geological conditions are known, and if no limit is placed on cost. In practice, the construction of a dam involves compromise and adjustment between these two extremes. The one great point to be emphasized is that engineers should have before them as complete and reliable a presentation of the geological conditions as it is possible to obtain. That lesson, no one who has seen the ruins of the St. Francis Dam is ever likely to forget.

CONSTRUCTION METHODS AND PLANT LAYOUT AT COOLIDGE DAM, IN ARIZONA

By J. G. Tripp,[10] M. Am. Soc. C. E.

Introduction

In planning his procedure for building the Coolidge Dam on the San Carlos Project, Arizona, the contractor was faced with unusual difficulties as to plant selection and the choice of handling methods. There were three main groups of problems involved in laying out the construction plant: (1) Concrete; (2), excavation; and (3) general service problems that applied to both excavation and concrete.

Problems relating to the concrete plant involved the selection of chutes, towers, bunkers, mixers, "aggremeters", cables, anchors, steel fenders, cutters, carpenter shop machinery, and many special small tools required in the construction of the dam.

Excavation problems included the choosing and arrangement of equipment necessary to control the Gila River during construction; to excavate and dispose of 300 000 yd. of rock and 50 000 yd. of sand, gravel, and boulders at the dam site; to load and transport the concrete aggregates; and to build construction roads, etc.

In the third group was included the plant and equipment that served the general needs of both concrete and excavation activities. It involved the construction and equipment of shops for mechanical repairs and for blacksmithing, welding, rigging, etc. It also included the erection of cableways and the establishment of systems for electric power distribution, flood lighting, telephone and signal communication, water supply, compressed air supply, garbage disposal, sanitation, etc.

The selection of plant machinery was guided by careful attention to the following factors: (1) The shape of the canyon floor and walls; (2) the general topography of the dam site; (3) the total quantities involved; (4) the element of speed in construction; (5) specification limitations; (6) the kind and the cost of labor available; (7) the "construction symmetry" of the design; (8) the kind of power available—whether electric, steam, Diesel, gas, etc.—and the cost of power; (9) relative economies and first costs; (10) operating and probable salvage value of the plant; (11) transportation facilities, including space available at the site, freight rates, probability of floods, and climatic hazards in general; and (12) general economic considerations.

It is conceded that the constructor must make a profit in order to remain in business, and, therefore, a strong undertone of economy must influence his judgment in the selection of his operating plant. Equipment with a probable life equal to or less than the length of the job must be selected on the basis of first cost. Other equipment must be purchased with consideration as to its possible use on other work as well as its final salvage value.

---

[10] Contr., Los Angeles, Calif.

LAYOUT OF THE CONCRETE PLANT

Since the Coolidge Dam is located in a "box" canyon (see Fig. 1), the constructor was forced to choose between two possible layouts for his concrete plant; that is, it could be established in the river bed or on the hillside.

General Considerations.—The latter was chosen because the Government had constructed an ingress highway along the top and because the Gila River was known as a dangerous, "flashy" stream. Any of the construction plant placed in the river bed would be subject to damage and possible annihilation by floods.

There was only one location available for storing aggregates and this was ½ mile down stream from the dam. There were three possible ways of transporting this material to the site: By road or railroad running close to the side walls along the bottom of the canyon; by motor trucks on a long, serpentine, up grade; and by means of an aerial tramway.

By choosing the latter, the flood hazard and the cost of difficult hauling by trucks were eliminated. Furthermore, the operating economy of the aerial tramway overbalanced the first cost and made this system appear as the most desirable.

Distribution of Concrete.—After these general selections had been made it became necessary to locate, in detail, all the various units of the concrete plant, giving due weight, one by one, to each of the eleven factors previously mentioned. Since the shape of the canyon is boxlike with sharp walls, and the width of the river bed is nearly equal to the length of the crest of the dam, it was evident that any distributing plant would have to be placed so as to reach out along the axis fully as far at the bottom of the canyon as at the level of the crest. The shape of the structure was such that its length along the axis of the stream was approximately equal to its width across the canyon at the bottom. This forced the designers of the distributing plant to consider ways and means of reaching every part of the area at the base of the dam. If the canyon had been V-shaped, with a constricted waterway, the type of plant necessary for distributing concrete at the bottom would have been very simple.

At the top, the dam recedes more than 200 ft. from its up-stream contact at the foundation. Therefore, at the river foundation, the selection of the plant was dependent on the shape of the structure. In Fig. 3, the "coverages" or the areas covered by the various chutes are shaded. Considerable thought was given to provide flexibility so that the constructor could place concrete at points widely separated between the up-stream and the down-stream limits at the base and at the top.

Construction Symmetry.—After analyzing the plans carefully, the constructor found that one type of plant would be satisfactory for building the lower and middle parts of the dam, but would be of little practical use at the top of the dam. The solution of this problem involved a consideration of Factor (6), "Construction Symmetry of the Design".

It was decided to pour the massive parts of the dam with one type of plant and the thin sections with another. This involved an added expense due to the first cost of the extra equipment, but the type of plant selected for the

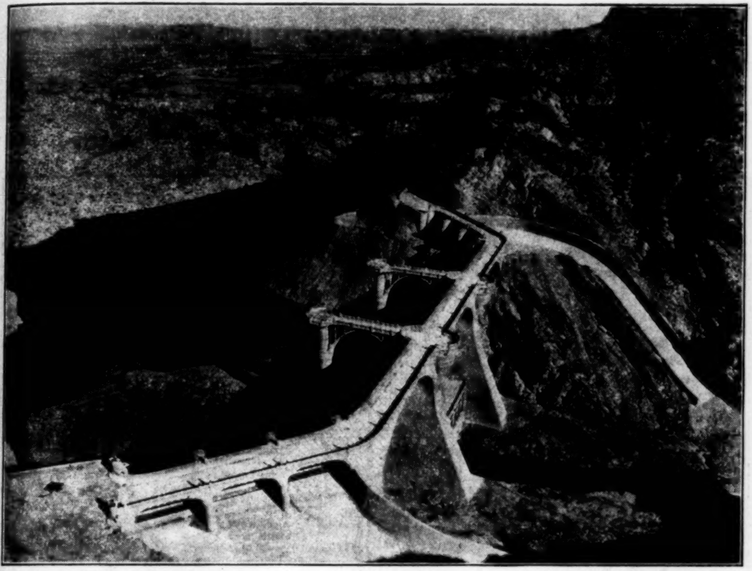


FIG. 1.—VIEW OF COOLIDGE DAM FROM POINT 300 FEET ABOVE THE CREST, JANUARY 19, 1929.

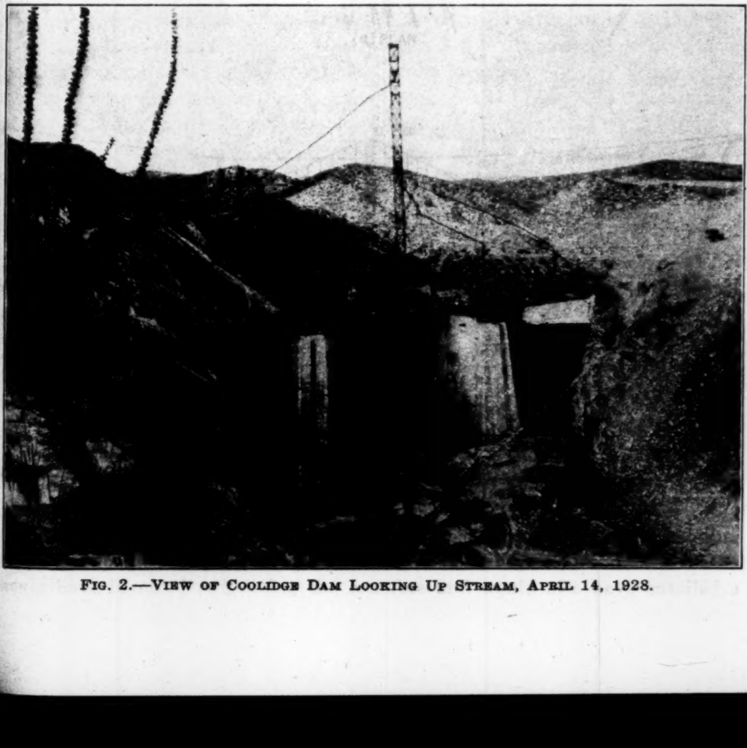
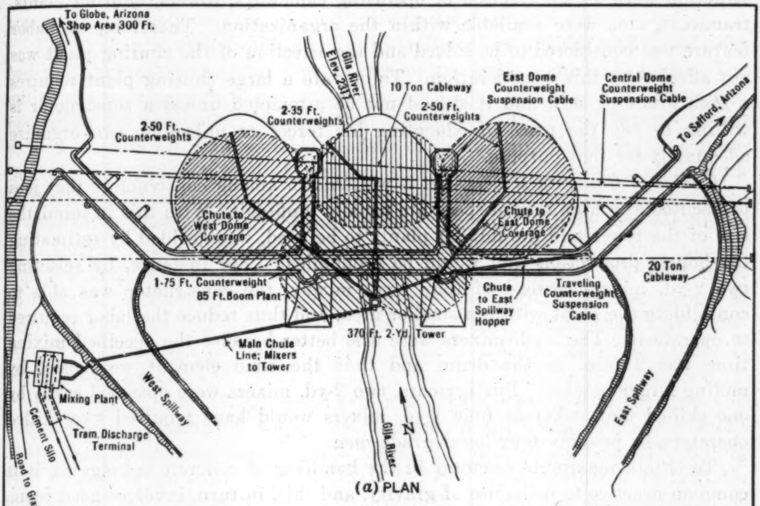
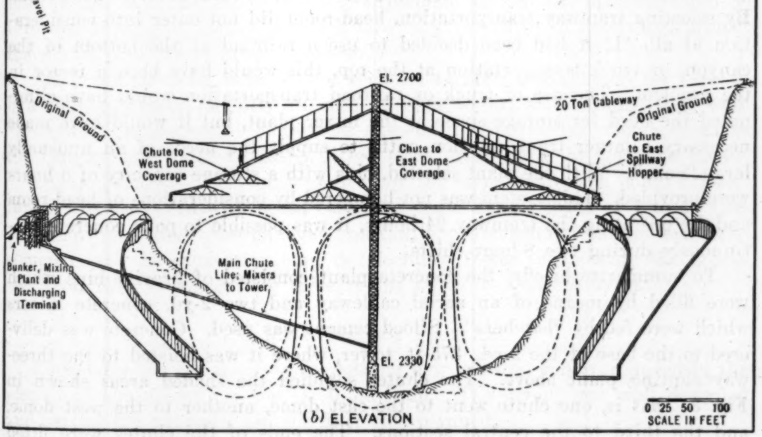


FIG. 2.—VIEW OF COOLIDGE DAM LOOKING UP STREAM, APRIL 14, 1928.

top work had some features that created economies which partly offset this handicap. Analysis of the plans showed that it was possible to place 2 000 yd. of concrete per day in many of the lower courses, whereas in some places at the top, it would be impossible to pour more than 100 yd. per day. One type of plant could not be operated economically under both these conditions.


To Globe, Arizona
Shop Area 300 Ft.
Gila River
Elev. 2317
10 Ton Cableway
East Dome Counterweight Suspension Cable
Central Dome Counterweight Suspension Cable
To Safford, Arizona
2-35 Ft. Counterweights
2-50 Ft. Counterweights
2-50 Ft. Counterweights
Chute to West Dome Coverage
Chute to East Dome Coverage
20 Ton Cableway
1-75 Ft. Counterweight
85 Ft. Boom Plant
Chute to East Spillway Hopper
Traveling Counterweight Suspension Cable
370 Ft. 2-Yd. Tower
Main Chute Line; Mixers to Tower
West Spillway
East Spillway
Mixing Plant
Cement Silo
Tram. Discharge Terminal
Road to Gravel Pit
N
Gila River

(a) PLAN


El. 2700
20 Ton Cableway
Original Ground
Original Ground
Chute to West Dome Coverage
Chute to East Dome Coverage
Chute to East Spillway Hopper
Bunker, Mixing Plant and Discharging Terminal
Main Chute Line; Mixers to Tower
El. 2330
0 25 50 100
SCALE IN FEET

(b) ELEVATION

FIG. 3.—LAYOUT OF CONCRETE PLANT FOR COOLIDGE DAM.

The Main Hoisting Tower.—Despite the flood hazard, the main hoisting tower was located directly in the river bed. (See Fig. 2.) Side-hill towers would have involved re-hoisting the concrete and would also have entailed a great first cost. In order to make the tower safe against high water, it was founded on a concrete pedestal which was sunk to bed-rock, and although it was not called upon to withstand a large flood, it probably would have been safe in the largest known run-off.

Labor Problems.—All the trained experts that were required for such construction work as the erection of operating cableways, towers, chuting plants, tramways, etc., were available within the organization. Therefore, the labor feature was considered to be solved and the selection of the chuting plant was not affected by this consideration. To operate a large chuting plant requires a high grade of skill and it should not be attempted unless a constructor is willing to pay the price of educating his forces or unless he can organize adequately for that purpose.

Choice of Concrete Mixers.—It is well-known among constructors that jobs prosecuted at a high rate of speed are usually profitable. With this in mind, the size of the tower and the mixing plants to be used, was decided by estimating the hourly production that would be economical. For instance, by selecting two 2-yd. mixers instead of four 1-yd. mixers, the constructor was able to consolidate the plant within a smaller area, and thus reduce the labor required to operate it. The 2-yd. mixers were also better because the specified mixing time was 2 min. in the drum and thus the time element was offset by mixing larger batches. Furthermore, two 2-yd. mixers were operated easily by one skilled man, whereas four 1-yd. mixers would have required two skilled operators, or possibly four lower grade men.

To obtain maximum economy in the handling of concrete aggregates, it is common practice to make use of gravity, and this, in turn, involves head-room. By selecting tramway transportation, head-room did not enter into consideration at all. If it had been decided to use a railroad at the bottom of the canyon, or truck transportation at the top, this would have been a factor in the problem. The use of truck or railroad transportation would have eliminated the need for storage space at the mixer plant, but it would have made necessary a larger transportation outfit to supply the needs of an unusually large "pour". With the plant selected, bins with a storage capacity of 8 hours were provided. This system was not hampered by considerations of head-room and, by operating the tramway 24 hours, it was possible to pour concrete continuously during two 8-hour shifts.

To summarize briefly, the concrete plant consisted of gravity bins which were filled by means of an aerial cableway and two 2-yd. concrete mixers which were fed by "batchers". Siloed cement was used. Concrete was delivered to the base of the 2-yd., 375-ft. tower, where it was hoisted to the three-way chuting plant above. The chutes supplied the shaded areas shown in Fig. 3; that is, one chute went to the east dome, another to the west dome, and the third to the central sections. The ends of the chutes were fitted with swinging counterbalances, as shown in Fig. 3. The 75-ft. machine at the center was used either by itself or in connection with two 35-ft. counterbalances that served the center dome. All swinging counterbalances were suspended from cables that stretched across the canyon.

The spandrel walls of the domes, the sidewalks, hand-rails, enrichments, etc., were poured by means of an automobile hopper car on a timber track. A total of 204 000 cu. yd. of concrete of all classes was used.

THE EXCAVATION PLANT

In planning for the excavation work, several selections were open to the constructor without danger of making any fundamental error.

The Transportation Problem.—The Gila River has a wide, sandy, and silty bed that carries the normal flow in one or two well-defined channels. The entire bed is subject to filthy inundation on almost any occasion. Certainly, one factor the constructor must secure is mobility. He must plan his layout so as to remove his plant quickly in time of flood. Since mobility is a function of size a steam shovel of medium size was chosen for excavation work in the river. At the same time, the high rock cliffs at each abutment offered a splendid opportunity for the use of large shovels. These two conditions were incompatible: On the one hand, the total plant investment must be kept low and, on the other, mobility and service at any required point must be assured. Therefore, it was decided to provide 1¼-yd. units and to use as many of them as was economically possible. Truck rather than rail transportation was chosen because of greater mobility and the ability of trucks to climb heavy grades.

The Flood Control Problem.—There was always some flow in the river and only a small quantity of water was required in the basin directly in front of the dam in order to create a high head. To meet this condition a sliding type of control gate, 12 by 32 ft., was built to close against a hydrostatic head of 150 ft. The gate was made adjustable so that the opening could be throttled to control, spread out, and minimize the effect of floods and protect the gravel plant from excessive water.

All pumping units were of the vertical, turbine type, chosen for ease in handling, for their mobility in case of high water, and because they were "fool-proof" in general. They were suspended from cables stretched across the canyon and could be easily lifted away from harm by gasoline hoists.

Dump Trucks.—The trucks chosen for the river excavation could "track" forward or backward with equal ease, and could handle loads of 7 to 10 tons on dump roads that were changed constantly. "Backdown" ramps were constructed into the excavation, and the deciding factor in choosing the trucks was their ability to negotiate these ramps safely by backing down.

GENERAL SERVICE PROBLEMS

Except for the aerial tramway, nothing new or novel was introduced in the plant and equipment for the Coolidge Dam. The layout was well designed and served the purposes for which it was planned.

Crushing and Screening Plant.—After selecting and locating the best type of mixing, chuting, and gravel transportation for the job, the next consideration was to erect a crushing and screening plant that would fulfill the requirements of the pouring schedule as planned.

The material in the gravel pit consisted of cobbles, boulders, silt, and mud, and it was, therefore, not possible to use it in its natural state as graded aggregate. It was necessary to furnish two digging units and a system of balanced transportation between these units and the primary crusher in order to overcome this feature.

The selection of equipment for the pit transportation plant was governed by ordinary economics and it was on this basis that a narrow-gauge track and "dinky" locomotives were chosen. The length of haul was the deciding factor in eliminating trucks. The digging units were chosen merely on the basis of judgment as to their adaptability to the particular kinds of ground to be excavated. A comparatively large shovel was used for the major digging operations and smaller ones for the secondary or "paging" work.

The gravel plant requires no further description than to state that it served all the purposes for which it was built.

Cableways.—The greatest care was taken in assembling the plant designed for "general services". Two inlet towers were located in the plane of the up-stream bed-rock contact. These towers received material during construction by means of a 10-ton cableway, which was also arranged to carry forms and reinforcing steel to the lower areas of the domes.

Most of the tonnage was delivered on the main 20-ton cableway (see Fig. 3), which was centered over the roadway of the dam. This material-handling service was augmented by several small trucks and one large truck which shuttled up and down the roadway between the camp, shops, and the bottom of the canyon.

Shops and Service Pipe Lines.—Two compressors fed a generous supply of air to the whole job. The power distribution was carefully planned and scrupulously maintained. The Electrical Department was completely equipped. An ice manufacturing plant was part of the layout and cold drinking water was supplied at various points on the job.

Air and water supply pipes were incorporated in all parts of the dam so that the forms could be readily cleaned and the concrete cured. The forms required were unique and the work entailed in erecting them was indeed a pioneering effort.

THE CONSTRUCTION WORK

In brief, the construction of the dam required a combination of old ideas with newer methods. It may be considered as comprising three distinct classes of work: The buttresses, the domes, and the "top work", which was quite ornamental.

The Buttresses.—The construction of the buttresses presented no problems, except that it was complicated by the need of providing the engineers with a leveling control for the sloping "pours". This involved additional costs in shoring and in wires, but otherwise there was no difficulty.

The Domes.—These are enormous egg-shaped structures, rising in nearly vertical lines from the bed-rock to a plane about 50 ft. above. At this plane, the surface began to incline inward toward the vertex with the curvature gradually increasing until at the top the crown line was level.

The total horizontal distance from the up-stream bed-rock contact to the down-stream face at the top of the dam is 212 ft. and the total height about 245 ft. The walls were 20 ft. thick at bed-rock and tapered to 4 ft. at the top. The span of the domes increased from 94 ft. at bed-rock to 160 ft. at mid-height and then decreased gradually to 126 ft. at the top. The rises were 30 ft. at the bottom, 50 ft. at the center, and 24 ft. at the top.

For purposes of construction each dome was divided into three sections: The part from bed-rock to a plane 50 ft. above; that between Elevation 50 to a level near the top, where the spherical cap began to take shape; and, finally, the top.

To guide in setting the forms for the domes, survey points were set at each 4-ft. elevation. Therefore, the concrete would have to be set hard enough by 9:00 A. M. each day so that the surveyors could set new points promptly. Otherwise, the cycle would be broken and one working shift would be lost. This stipulation required that the yardage of concrete poured each day should be such that the plant could be used to full capacity and that the chuting system be made so flexible that the scattered areas could be easily reached.

It was found best to pour the buttresses and the quarter-spans of the adjacent domes in one piece. After such a section had been allowed to shrink 4 or 5 days, the central part of the dome was poured. This plan accomplished two things: It permitted pouring concrete in the buttress sections with very little moving of chutes, etc.; and it localized the sections of scattered pouring to the central arch parts of the domes. The flexibility built into the plant was made usable during the time of concentrated pouring so that a balanced yardage and form area were maintained.

Since the walls of the domes were of varying rates of curvature, flexible form panels, in assorted sizes, were used throughout. No difficulty was experienced until the over-hang of the intrados panels became greater than 35° from the vertical. To overcome this, reinforced steel tie-backs were set in each successive lift of concrete to hold the form in place. By means of these tie-backs the dome was poured nearly to the top and until the over-hang approached 54° from the vertical.

The caps were divided into two parts: That which lends itself to bridge treatment and another part, named "the wart", for which a new and novel treatment was devised.

The bridge, or roadway part, of the domes was supported on structural steel centering. The concrete was poured in balanced blocks in such a manner as to control deflections. Well-known and standard methods were used.

Steel U-bolts were cast into the bridge barrel to match a similar number cast into the last section poured in the dome where the degree of curvature was 54° from the vertical. In each dome, twenty-two bowstring trusses were suspended from the U-bolts, and the lagging to support the concrete for the top of the domes was constructed on these trusses. The loading on the trusses was distributed so that each section poured became self-supporting before a new load was applied. This method was found to be economical and provided for the necessary shrinkage in the concrete.

The pouring process on each "wart" took three weeks. No difficulty was encountered in their construction, but the design of the centering required all the ingenuity that could possibly be concentrated upon it. It was necessary not only that the superimposed loads on the trusses be carried safely and that the deflections be controlled, but also that over-stress be avoided in the parts of the incompleted domes used for supporting the trusses. This involved considerable "cut and try" designing.

The multiple dome was found to be a practical structure to build and a worthy contender for primacy in the high dam field.

DISCUSSION

F. G. Switzer,[11] Assoc. M. Am. Soc. C. E. (by letter).—In reading the interesting paper by Mr. Wiley, three points were noted with which the writer cannot agree. These may be summed up as follows:

First.—If a gravity dam is designed as a straight dam with the resultant line of pressure at the third point, then the same section built as an arched gravity dam will show no tension.

Second.—Arch action is always present in an arched gravity dam.

Third.—There can be no vertical stress in the concrete as a result of water, under pressure, in the voids.

The writer believes that the first and third of these points are contrary to fact and that in the second point the word, "always", should be replaced by the words, "not often."

As a matter of experience, the writer recalls an existing gravity dam, for which the first designs were those for a straight dam. The contours in the river bottom made it desirable to arch the dam in plan in order to utilize the best abutment sites and, at the same time, avoid a deep hole in the river bed. With the same section, computations showed that in the section reduced by convergence of the radii, tension would be present for some appreciable distance from the up-stream face. This design assumed full uplift pressure on the whole area at all elevations and with a maximum intensity equal to the full hydrostatic head. The section, therefore, was increased, and no reliance was placed upon possible arch action.

In order to examine somewhat more closely into the problem, the writer has considered the simple triangular section of a gravity dam, as shown in Fig. 4. The reduction in section due to convergence of the radii has been expressed by a fraction, k, which is the ratio of the base length of the dam, b, to the radius of curvature of the up-stream face, R. If the section has a length across the up-stream face of 1 ft., the toe of the section has then a length across the river of $(1 - k)$ ft. In order to simplify the work, the water level was taken at the top of the section, and no uplift pressures were assumed. The base length was determined from the consideration that the resultant pressure with a straight dam, $k = 0$, should intersect the base at the down-stream third point. Thus,

$$b = h \sqrt{\frac{w}{\rho}} \qquad \text{(1)}$$

in which, w is the weight of water per cubic foot, and ρ is the weight of the concrete per cubic foot. Computations were then made to determine the stress intensities at the up-stream face of the dam for any value of k and, hence, for any radius of curvature. Without going into the details of the mechanics and algebra, the stress intensity, S, at the up-stream face, when considered as a vertical cantilever, can be found from Equation (2):

$$S = -\frac{\rho\, h\, k\, (2 - k)}{6 - 6\, k + k^2} \qquad \text{(2)}$$

---

[11] Cons. Engr. and Prof. of Hydr. Eng., School of Mech. Eng., Coll. of Eng., Cornell Univ., Ithaca, N. Y.

The negative sign denotes a stress which is tensile for any degree of convergence as expressed through values of k between zero and 1. Let $\rho = 150$ and $h = 100$; then Equation (2) may be solved for S by introducing successive values of k, as follows:

k.	S, in pounds per square foot.
0	0 tension
0.1	526 "
0.2	1 116 "

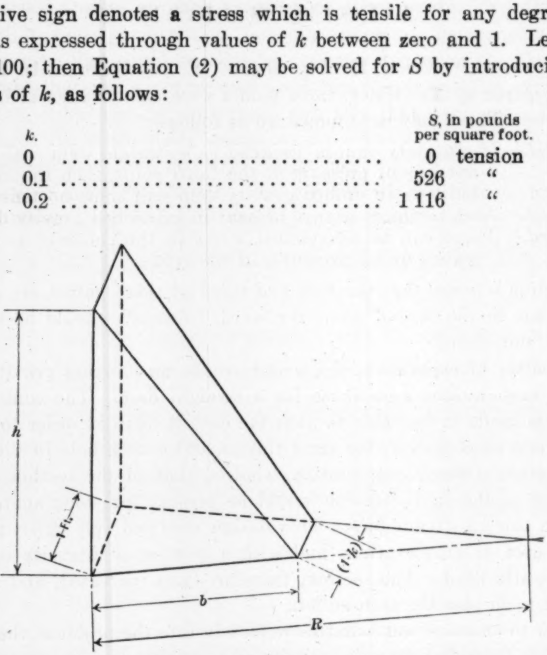


FIG. 4.—TRIANGULAR SECTION OF A GRAVITY DAM.

As for the presence of arch action, the writer has estimated the deflection of the section shown in Fig. 4 when considered as a vertical cantilever on unyielding foundations. Let E, for concrete, $= 3\,000\,000$ lb. per sq. in., $\rho = 150$ lb. per cu. ft., and $h = 100$ ft.; then, the top of the section deflects down stream about 0.032 in. With such a small deflection the convergence of the radii would not close up any contraction joints such as are used ordinarily between blocks of the dam, and, therefore, no arch action can develop. Arch action can be expected only when the contraction joints between blocks are carefully grouted at a time of low temperature or when the arch has a comparatively short radius of curvature. Since these conditions are not often found, it may be said that arch action in a curved gravity dam cannot be expected.

Finally, the writer would disagree with Mr. Wiley's reasoning which leads him to conclude that "there is no uplift in the body of a homogeneous mass of concrete". If a section of the dam above any given elevation be removed, in order to represent it as a "free body", according to the usual methods of mechanics, the plane of separation will cross voids as well as solid material. If the voids contained water under pressure, the severance of the structure requires that pressures equal to the water pressure be exerted upward where these voids existed. This is the uplift pressure. The opposite and downward pressures are exerted upon the section of the structure below the line of separation and not on the upper section. If the voids contain water under pressure, then there will be uplift forces tending to create tensile stresses in the solid surrounding the voids and thus in the section of severance. Mr. Wiley's observation of the seepage of water from the body of the St. Francis Dam for weeks after the failure, must be considered as further conclusive evidence that water under pressure does permeate the voids in concrete dams.

FRED A. NOETZLI,[12] M. AM. SOC. C. E. (by letter).—In the design and construction of dams of ever-increasing height many new problems are encountered, some of which are of vital importance for the safety of such structures. If the law of similarity would hold in every respect, then, within proper limits, high dams built on an adequate foundation should be as safe as low ones.

Unfortunately, certain conditions have the tendency of making high dams relatively less safe than lower structures of the same type. Attention is called, in this connection, first, to the danger of buckling the face slabs of high earth or rock-fill dams due to a relatively larger settlement of the fill of the higher dams; second, to the complicated stress conditions and the tendency of tension cracks developing in the thick arches of high arch dams; and, third, to the tendency of cracks developing in the buttresses of high dams of the buttressed type, such as multiple-arch, Ambursen dams, etc. Only recently it was observed that shrinkage cracks have also occurred in the interior of high concrete gravity dams. While such interior cracks may not be serious in case of relatively low gravity dams in which the shear stresses are small, the stability of high gravity dams may be endangered if the cracks should extend in the direction of high shear stresses.

Large concrete dams of the gravity type, if built rapidly, develop high temperatures in the interior of the mass. Later, when the internal heat is being dissipated, the concrete shrinks and, being restrained along the rock foundation, is setting up tension stresses which may exceed the tensile strength of the concrete and produce cracks. In order to avoid such cracks, vertical contraction joints, usually about 50 ft. apart, are being provided in practically all concrete gravity dams. Not until after the failure of the St. Francis Dam was the definite proof available that similar cracking was likely to occur also in planes parallel to the axis.

In Fig. 5 are indicated some of the cracks which were observed in the remaining part of the St. Francis Dam after the failure. Water seeped out for several days after the failure, indicating that the cracks extended over considerable areas and were subjected to internal water pressure. A gravity dam, say, 250 ft. high, may be about 170 ft. thick at the base. There is no reason why the concrete should not shrink in an up-stream and down-stream direction the same as in the direction of the axis of the dam. Thus, in the 170-ft. thickness of the dam, the shrinkage of the concrete will probably produce cracks unless contraction joints are provided at suitable intervals. There is no other alternative unless new materials or construction methods are developed which will furnish concrete that does not shrink. As Mr. Henny pointed out, "such cracks are likely to be approximately vertical and are thus often close to the direction of maximum shear."

---

[12] Cons. Hydr. Engr., Los Angeles, Calif.

Fortunately, the theory of the principal inclined stresses offers safe and adequate means for avoiding irregular internal shrinkage cracks by providing inclined contraction joints parallel to the direction of the principal stresses for maximum load. In these planes the shear is zero. Consequently, there is no tendency for a relative movement to take place in the parts of the dam on opposite sides of such joints. Contraction or expansion of the concrete may take place by opening or closing of the joints and with little or no effect on the load stresses in the dam.

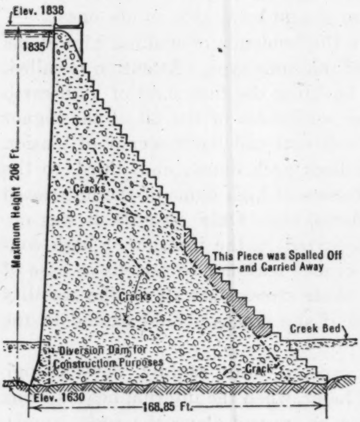

Elev. 1838
1835
Maximum Height 206 Ft.
Cracks
This Piece was Spalled Off and Carried Away
Cracks
Creek Bed
Diversion Dam for Construction Purposes
Crack
Elev. 1630
168.85 Ft.

FIG. 5.—MAXIMUM SECTION, ST. FRANCIS DAM, SHOWING CRACKS.

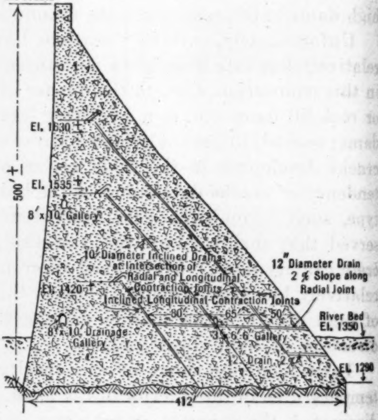

El. 1782.5
El. 1630 ±
500' ±
El. 1535 ±
8' x 10' Gallery
10' Diameter Inclined Drains at Intersection of Radial and Longitudinal Contraction Joints
12" Diameter Drain 2 % Slope along Radial Joint
El. 1420 ±
Inclined Longitudinal Contraction Joints
80'
65'
50'
River Bed El. 1350
8' x 10' Drainage Gallery
3' x 6' 6" Gallery
12" Drain 2 %
El. 1290
412'

FIG. 6.—MAXIMUM SECTION, SAN GABRIEL DAM, SHOWING INCLINED JOINTS.

Fig. 6 shows the location of the three longitudinal joints in the cross-section of the San Gabriel Dam. The joints are offset in the sections on opposite sides of the vertical radial contraction joints. Interlocking keyways will prevent excessive movements of any portion relative to the adjoining portions so that the whole structure will act as a system of inclined columns, each of which is thoroughly interlocked with the adjoining columns. The inclined columns have a horizontal cross-section of approximately 50 by 60 ft. They may be built up individually for several lifts ahead of the adjoining columns so that at least a part of the shrinkage can take place before others are poured to the same elevation. Also, the individual pours may be interlocked with each other. In this case the inclined joints would have offsets of perhaps 6 to 12 in. each at intervals of, say, three or four lifts. If these offsets are troweled smoothly and painted with oil to prevent bond, a slight lateral movement corresponding to the shrinkage of the concrete could take place and thus the inclined joints made effective without material sacrifice of the strength of the structure.

In this connection attention is called to the San Mateo (concrete gravity) Dam built in 1887 and 1888 for the water supply system of San Francisco, Calif. This dam was constructed of blocks of concrete, 10 to 12 cu. yd. each, cast in place and thoroughly interlocked with each other. If the theory of the principal inclined stresses in gravity dams had been known at that time, and if the designer had arranged the up-stream and down-stream slopes of the individual blocks in continuous lines parallel to the direction of the principal stresses, this dam would be representative of the most advanced type of construction. Apparently, the designers and builders of the San Mateo Dam anticipated by sheer ingenuity or intuition a requirement in the construction of concrete gravity dams which only recently was substantiated by theoretical deductions. It can no longer be ignored without inviting possible disaster, especially in the case of high dams.

Similar conditions as to shrinkage cracks obtain in the buttresses of high multiple-arch and Ambursen dams. Cracks have been observed in the buttresses of many of these structures. Fortunately, in most cases they appear to have developed under load conditions, and their direction was guided by the laws of Nature to be approximately parallel to the principal stresses. Two engineering commissions recently investigated the Lake Hodges Dam in California. Both reported that the various inclined cracks, of which there is at least one in practically every buttress of the dam, did not render the dam unsafe. This conclusion seems to be substantiated by the fact that the Lake Hodges Dam, built in 1917, has since been filled several times. The buttresses contain practically no steel reinforcement; nor is any reinforcement apparently contemplated despite the presence of the cracks, some of which are nearly 0.1 in. wide and extend from the base of the buttresses forwardly inclined to practically the groin line of the arches.

The cracks in the Lake Hodges and other dams are evidently a good and sufficient proof of the feasibility, not to say, necessity, of providing large concrete dams, both of the buttressed and of the gravity type, with inclined contraction joints. The agreement of these "full-size tests" with the theory of the principal stresses is quite satisfactory and is further subject to verification by constructing in the ordinary way the force and string polygons for water load and weight of the individual parts of the dams separated by the joints. This was done in the designs of the Coolidge (multiple-dome) Dam, completed in 1929 for the U. S. Department of the Interior, Indian Service, on the Gila River, in Arizona, and for the San Gabriel Dam.

Mr. Wiley's statement that, in his opinion, "the higher dams of the future will be of a solid masonry gravity type," is somewhat startling. As a matter of fact the two highest dams built so far, namely, the Diablo Dam on the Skagit River in Washington, 385 ft. high, and the Pacoima Dam in California, 372 ft. high, are of the single arch type. There is under construction in France an arch dam which, when completed, will be 440 ft. high. This dam is located in the extremely narrow canyon of the Drac River in the French Alps and is only 55 ft. thick at the base.

The Owyhee Dam, 405 ft. high, mentioned by Mr. Wiley as being of the arched gravity type, was designed and is being built as an arch dam.[13] Furthermore, studies are being made to determine whether the proposed Boulder Canyon Dam, 680 ft. high, can not be designed to act as an arch, at least in the upper portions, thereby decreasing the enormous quantity of concrete required for a pure gravity dam while at the same time increasing its safety factor against overturning and sliding. Thus, it is seen that the highest dams so far built or under construction are not of the gravity but of the single arch type.

---

[13] *Western Construction News*, January 10, 1929, p. 9.

Dams of the buttressed type, after cautious beginnings some time about 1910, have already been built to a height of 250 ft. There is no reason why dams of this type should not be built with perfect safety to still greater heights. Undoubtedly, this will be done sooner or later.

A gravity dam has a true factor of safety of less than two. This is less than for any other type of engineering structure. Furthermore, such a dam is subject to uplift pressure and to internal tension stresses of unknown, but certainly of large size, sometimes resulting in cracks such as have been observed in the interior of a number of dams of this type. While these cracks appear to be of small consequence in case of relatively low dams, because the shear stresses are small, they are of increasing importance for high dams especially if, as Mr. Henny pointed out, the cracks should occur in planes of high shears.

It is generally believed that the design of a gravity dam is a simple matter and permits of an easy determination of the stresses. Recent investigations have shown, however, that a scientific design of a high gravity dam is a fairly difficult problem; at the same time it involves relatively large uncertainties, especially as regards the applicability of the law of the trapezoid for the stresses in thick dams, uplift pressure, internal temperature, and shrinkage stresses, etc. While some of the same difficulties are encountered in the design of high buttressed dams, the tension stresses in the buttresses may be taken care of by steel reinforcement; a similar procedure is not economically feasible in the case of gravity dams.

From these considerations and in view of the recent construction of numerous high dams of "less than gravity" section, does it not appear that there are strong tendencies toward building future dams of other than the concrete gravity type?

Edward Godfrey,[14] M. Am. Soc. C. E. (by letter).—The entire subject of dams, in the writer's judgment, needs to be thoroughly and freely discussed. The design of dams will never be on a sound footing until the strenuous objections to standard methods of a small, very small, but growing minority of engineers are met and clarified.

The writer is one of those who hold that standard design is woefully inadequate and has been responsible for practically all the large number of failures of masonry dams. The one great and serious fault can be simply stated: It is neglect of under-pressure. That it is in truth standard to ignore under-pressure in the design of dams can be proved in many ways. In the first place very few books on the design of dams mention or give any prominence to under-pressure. Many of those that do mention it, treat it as if it were merely a refinement—something for the designer's judgment. Some writers even refer to it as a bugaboo.

---

[14] Structural Engr., Pittsburgh, Pa.

That it is standard to neglect under-pressure in designing dams is also shown by the statement of Mr. Wiley that in many, if not most, straight gravity dams, no allowance has been made for uplift.

There has scarcely ever been a failure of a masonry or concrete gravity dam where the entire action of the dam cannot be fully explained by supposing that water under pressure worked its way under the dam or into a horizontal joint and lifted the dam, allowing it to float away or to be overturned; and there has never been a dam lifted and pushed down stream where the mass and base width were sufficient to hold it in place against assumed up-stream and under-pressure. Unless the truth of these statements can be denied, they stand as an unanswerable argument for unconditional provision for under-pressure and against standard methods that ignore under-pressure under any condition whatever. It is no answer to state a contrary opinion. The only refutation of these statements of any value is the citing of one or more dams the failure of which can not be explained by under-pressure, or of one which floated off, as did the St. Francis Dam, although the base width was 85% of the height instead of 65% as was that dam.

In reading Mr. Wiley's paper one would be led to believe that under-pressure is of little consequence and internal pressure, even in porous concrete, is non-existent. From Mr. Henny's paper one would be led to believe that under-pressure is easily averted or avoided.

Mr. Wiley states that:

> "The absence of stress due to uplift in the body of the concrete appears from the consideration that the water can act only in the voids. The pressure in a void in the concrete is in all directions, and considering only the vertical pressures in a single void, it might appear that a tensile stress in the concrete would result; but considering that the upward and downward forces in each void are opposed and cancelled by equal downward and upward forces in the voids immediately above and below, it is readily seen that the summation of the vertical forces in the voids is zero, and that no vertical stress in the concrete can result from water pressure in the voids. This has also been shown by the application of enormous pressures to a liquid without effect upon the concrete or other porous substance immersed in it."

This is a new line of reasoning which, if valid, calls for a recasting of the theory of pressures. It would mean that a liquid could be carried slowly through a spongy mass under high pressure without exerting tension on the pipe. It is easy to see that a spongy mass embedded in a fluid under pressure is not subject to tension in its own fibers by reason of the pressure; but, at the same time, if there is enormous pressure there is enormous tension somewhere that exactly balances it. The tension is not in the sponge but in the vessel containing and restraining the fluid. Suppose that the sponge itself is the vessel; suppose the outer skin of a sponge were elastic and impervious and that the sponge were filled with water under pressure. Every fiber of that sponge would be subject to tension. It requires no test to prove this. If there were a spongy layer in the concrete of a dam, and water entered that and was prevented from escaping on the down-stream face (and the caretaker would see to it that the joint would be sealed up), there would certainly be tension in the concrete, a tension exactly equal to the pressure. This is directly contrary to Mr. Wiley's statement of the case, but it is the law of fluid pressure and of mechanics.

Furthermore, this is just what happened in a remarkable case of the flotation of a pier[15] which split at a construction joint in the concrete. The upper part, weighing about 1 000 tons, was lifted by pressure of water that entered this joint in the seemingly solid concrete. The great weight of the pier and the friction against the coffer-dam, as well as what tensile strength the concrete possessed, were not sufficient to overcome the water pressure in the pores of this concrete, which, by Mr. Wiley's statement of the case, should be balanced and equal to zero.

The several means used to inhibit under-pressure in dams are: Cut-off walls, grouting fissures, and under-drainage.

Consider, first, cut-off walls. If water under-pressure has an outlet, the pressure will be reduced. If it has no outlet, the pressure will tend to reach that of the head, or the vertical distance to the free water surface. The distance through which the water and the pressure travel has influence only as affecting the time necessary to accumulate the full head. The value of cut-off walls is, therefore, simply to delay the accumulation of pressure. They do not prevent it, unless the water has a more or less free outlet.

The fact is that several dams have failed, that had one or more cut-off walls, but were too light for stability. Another fact is that pressures have been measured and found to be very considerable down stream from cut-off walls.[16]

If cut-off walls prevented pressure of water under a dam, there would never be a failure of a dam having a cut-off wall; because if the water were not in the foundation exerting pressure, even the standard reason for dam failure, according to all reports, would have no basis in fact.

Grouting fissures in a foundation is considered to be the one unfailing inhibitive against under-pressure. The basic idea behind this is that water can enter only where there is an actual fissure—an opening. A crack would not be classed as a rock fissure; and, furthermore, it would be quite impossible to grout up a crack. In fact, there are fissures that it would be practically impossible to fill with grout, even under pressure. Experience in grouting thin cracks has proved that the water of the grout will penetrate the crack and leave the cement. It stands to reason that solid cement or colloidal cement will not penetrate thin crevices where water may freely go. It is a well-known fact of hydraulics that water will exert its full pressure in the finest crevice.

Furthermore, it is not always possible to find all the rock seams. Some of these may be in a horizontal direction just beneath the base of a dam, and drilling would not disclose their presence. The cracks most difficult to grout are the horizontal ones, and these are the greatest menace to the stability of the dam.

In view of the facts just stated the stability of a dam that is dependent on the inhibiting of under-pressure because all rock seams are supposed to be fully grouted and impervious, is seen to be precarious.

---

[15] *Engineering News-Record,* March 13, 1904.

[16] *Transactions,* Am. Soc. C. E., Vol. 93 (1929), pp. 1532, 1537, and 1543.

The third method of avoiding under-pressure is drainage. This requires an expensive system of piping. There is no way to determine the effectiveness of the system. It is likely to silt up and be made useless. In cold climates the outlet is liable to freeze. The failure of more than one dam that occurred in cold weather was doubtless due to freezing of the weep-holes.

Water, working toward the drainage pipes under a dam, must of necessity exert pressure. This pressure will act against the base of the dam; hence, the only relief will be in the immediate neighborhood of the drainage pipes. Under-drainage is then, also, seen to be of precarious and unknown value.

Still another method used to prevent under-pressure is the driving of sheet-piling. This, too, has been found to be ineffectual in stopping pressure.[17] This method could be used only where the soil admits of driving sheet-piles, and such soil, not being solid and strong rock, would be classed by some as unsuitable for the foundation of a dam.

The evidence that under-pressure exists in many dams and may have a large value in any dam is overwhelming.[18] The possible pressure under the St. Francis Dam, for example, was hundreds of millions of pounds, which is a gigantic force to provide for, and a still more gigantic force to ignore.

It is pertinent to look into the arguments, the tests, and the ideas that influence some engineers to ignore under-pressure in the design of dams. The idea persists that, unless there is tail-water, there can be no flotation. This is born of the notion that, unless an object is immersed, it cannot float. Designers lose sight of the fact that it is in no sense the influence of water pressure on the side of an object that makes it float; it is solely the pressure of the water beneath it.

It is universally conceded that, where there is tail-water, there is flotation, or buoyancy equivalent to the loss of weight for the depth of that tail-water. It certainly is not pressure of water on the sides of the dam that is responsible for this buoyancy; it is, of necessity, pressure of the water beneath, and that water must work its way under the dam. There is greater facility for water and pressure to enter from the up-stream side than from the down-stream side because of the greater head.

One argument for adhering to the standard method of designing dams and ignoring under-pressure is that there are dams standing which are not proportioned for any under-pressure. The St. Francis Dam was in that class just before it failed. A large number of other dams that have failed stood in that class for as long as 40 and 50 years.[19] Some of these dams failed under a head of water lower than they had resisted in previous years.

A fact lost sight of is that it requires some time for water to penetrate and still more time for the pressure to build up. A high-water period may not last long enough for a destructive pressure to accumulate, and the false notion is fostered that the dam is stable. In other structures loads have an immediate effect, and a test load is conclusive.

---

[17] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 1537 and 1556.

[18] *Loc. cit.*, pp. 1317 and 1527.

[19] *Engineering News*, April 9 and June 4, 1903; *Engineering Record*, April 27, 1912, p. 476.

Furthermore, time has a controlling influence because the accumulation of silt may serve to clog the outlet where water escapes below a dam and thus allow the building up of pressure. The stopping of leaks below a dam, either by Nature or by the caretaker, is equivalent to driving a gigantic wedge beneath it.

The statement is sometimes made that water beneath and in a dam is present because of capillarity and that the dam is, therefore, not subject to pressure. It is well known that water leaks out below dams. If that is an exhibition of capillarity, there is a chance for some one to invent perpetual motion. Just suck up a lot of water to a high level by a system of sponges, and let it run away as free water to run turbines, and the question of power is solved.

It is argued that pressure on the bottom of an object (in sand, for example) can be only a fraction of the computed head because there is contact on part of the surface of a grain of sand where there could be no pressure. This argument ignores the fact that the next layer of sand grains will press against the one in contact and make up fully for any such contact.

Experiments have been made tending to show greatly reduced pressure by reason of contact in sand grains.[20] Strangely enough, this is attributed by some to voids in the sand. Such voids are a volumetric property; pressure is purely a superficial matter. There could be no relation between sand voids and pressure of water in those voids because pressure is totally independent of volume.

Tests of such properties made on only a few inches of water are unreliable, as many uncertain and unknown elements enter. For example, unless the small cylinders are plunged deeply into the sand, practically full head is shown by their rising out. In the deep position, however, the starting friction (augmented by pressure on top of the sand), air bubbles in the sand, and the time element for the accumulation of pressure, combine to show an apparent upward pressure of only a fraction of the actual head.

In the well-known process of jetting piles in sand one has only to apply the water at the bottom of the pile, and it will sink in. Let the sand settle around the pile, and it is very difficult to drive or to pull it.

Air bubbles and cohesion in the sand due to the presence of clay would have a measurable influence in tests where only a few inches of head is considered. A few inches of sand could be held together by these means and the lifting action would account for part of the head.

Tests of high heads have shown, in dams and dry docks and in laboratory experiments, that under-pressure is practically 100% after sufficient time has elapsed for the pressure to build up.[21] In the face of these actual practical tests it is foolish to set up others on miniature models, with all the attendant uncertainties already mentioned, and to design dams on the basis of these small tests, setting aside the proofs of large-sized models and actual experience of failures.

---

[20] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1334.

[21] *Loc. cit.*, pp. 1330, 1331, 1333, and 1354.

Pressure in so-called impervious soil, while it may take more time to build up, cannot be assumed to be less than in sand. The soil above and below the tunnels in New York, being silt and clay, would seem to be classed as impervious; but this soil shows most remarkable response to fluid pressure. It is observed that the tubes sink and rise as the tide rises and ebbs. This demonstrates, not only that this "impervious" soil transmits and exerts pressure, but the further fact that it requires some time for this pressure to travel through the mass; for if the pressure were transmitted at once, there would be no difference between the differential of top and bottom pressures at high and low tides, and thus there would be no movement of the tubes.

The writer believes that in practically every dam that has failed the reported reason has been "foundation failure"; but in no case, so far as he has been able to learn, have the successive steps of that foundation failure been described or analyzed. Excessive soil pressure due to weight cannot account for it, because that would merely make the dam sink. The earth cannot wash away unless there is a crevice through which water can flow; and, so long as the dam is in contact with the soil, there can be no such crevice.

The term, blow-out, is sometimes used to describe the failure. A blow-out of soil connotes high pressure, and high pressure under a dam is something non-existent, according to those who make this explanation and fail to see that that high pressure would lift and float a dam not proportioned to resist it long before it would issue in a blow-out.

It would be impossible for rock, no matter how poor a quality, to issue in a blow-out. Only fluid or semi-fluid soil could be forced out from beneath the base of a dam.

It is the foregoing arguments against the neglect of under-pressure that make it incumbent on the Engineering Profession to do one of two things, namely, either to answer these arguments in a free and open discussion or to revise their standards of designing dams.

Mr. Wiley mentions the economic advantage of arch action in dams. Reading his remarks, one cannot fail to gather the idea that merely curving the dam in plan makes an arch of it. Nothing is said of the abutment or the need of an abutment. A curved dam is merely a curved dam; it is not an arch, unless it has adequate abutments at the ends of the curves. Dams like the St. Francis Dam are not arches, for they taper down to practically nothing at the very place where gigantic abutments should be located. If suitable abutments for such long dams, not in rock gorges, were included in the design, the seeming economy would vanish. At least, two long, curved dams have failed because their designers considered them as arches, but supplied no abutments.[22] This is another of the major errors in standard dam design.

B. F. Jakobsen,[23] M. Am. Soc. C. E. (by letter).—In his paper Mr. Wiley defines arched gravity dams, as follows:

"In the case of the arched gravity dam there is a massive section capable of carrying the full load by its weight alone and, in addition, there is the resistance of the massive horizontal arch."

---

[22] *Engineering News-Record*, October 13, 1926, p. 616.

[23] Cons. Engr., Los Angeles, Calif.

This is, in the writer's opinion, a correct definition and the only logical one. It is quite important that engineers should agree on a definition for gravity dams.

Elsewhere in his paper Mr. Wiley lists the Exchequer Dam as an arched gravity type, although the writer has shown that this dam has a tension of about 40 lb. per sq. in. at the up-stream face, when concrete is assumed to weigh 152 lb. per cu. ft.[24] This dam, therefore, does not conform to Mr. Wiley's definition, since it will not stand by its weight alone, even when no uplift is assumed, and it should not be classed as a gravity dam. In discussing this subject further, Mr. Wiley states:

> "In all these and in many other dams of similar design, the convergency of the radii is assumed to be offset by arch action and the uplift reduced, by pressure grouting and drainage, to an amount which can be taken care of by arch action."

This statement does not conform to the author's definition. That an arched gravity dam must be treated as such and not as a straight dam, the writer has argued fully in a recent paper,[25] and he notes with satisfaction that Mr. Henny agrees with this and also includes uplift. The writer can only insist that the factor of safety of a dam should be a calculable quantity and not something which is assumed; and he cannot agree to assume that arch action in general has certain beneficial qualities, unless these can be shown to exist and their magnitude calculated. Likewise, the writer is unwilling to assume that arch action in unknown amount can offset an unknown amount of uplift, as is done by Mr. Wiley. To assume no uplift is to make the least conservative assumption possible and is not, in the writer's opinion, good engineering.

In discussing the arch section *vs.* the straight section, Mr. Wiley states:

> "* * * if there are no open joints extending entirely through it, the slightest deflection of the dam as a gravity cantilever will cause an equal deflection in the horizontal arch and a division of the load in some proportion between the cantilever and the arch."

This assumption overlooks the fact that before shrinkage cracks appear, the concrete must be in tension; and, in that case, a deflection of the dam as a gravity cantilever will not cause the arch to take any part of the load. Before this can happen, the cantilever must be deflected sufficiently for the arch to be relieved of all tension produced by shrinkage of the concrete. It is this fact and others, which makes the calculation of the division of load between the cantilevers and the arches so problematical. It has convinced the writer and other engineers that gravity dams should be designed as such, irrespective of arch action, and that arch dams should be designed as such, irrespective of any possible cantilever action.[26] This opinion was evidently shared by Mr. Wiley himself, who a few years ago wrote:[27]

---

[24] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), pp. 772 and 835; see, also, Mr. Wiley's discussion on p. 821.

[25] *Loc. cit.*, p. 766.

[26] "Stresses in Thick Arches of Dams," by B. F. Jakobsen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 516, Article XXI; see, also, the writer's remarks, *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 844.

[27] *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 116.

"It appears to the writer that in designing any arched dam of either full gravity, or reduced gravity, section all stresses should be considered as transmitted in either vertical or horizontal planes."

That arched gravity dams develop shrinkage cracks and leakage was brought out by the writer in discussing the Mulholland Dam[28] and E. W. Kramer, M. Am. Soc. C. E., recently stated:[29] "Some further reference to the Mulholland Dam will be made to prove that the contraction of concrete in a dam is real and not theoretical."

That dams not provided with contraction joints must develop cracks has been stated by Mr. Wiley,[27] as follows:

"Nearly all high dams * * * are of necessity provided with construction joints if not contraction joints. If they are not so provided, they are sure to crack into sections at points which cannot be determined in advance. It is impossible to build, or at least to maintain, a large dam as a monolith."

The writer agrees with this statement, but does not see how Mr. Wiley's Conclusion No. 3 can be made to agree with it.

In discussing uplift inside the dams Mr. Wiley appears to confuse external forces with internal stresses, and his attempt to reason uplift out of court does not carry conviction. Before it can be safely disregarded, it must be shown not only that it does not occur, but that it cannot occur in actual practice.

Under the heading, "Pressure Grouting of Contraction Joints," Mr. Wiley gives it as his opinion, that such grouting would be of no appreciable value and that the joints would only be filled for a distance of about 15 to 20 ft. from the faces of the dam. In making this statement, he probably overlooked the elastic property of concrete, since he mentions filling the contraction joints. Some years ago, L. R. Jorgensen, M. Am. Soc. C. E., suggested[30] pressure grouting of the contraction joints of single arch dams with a view of compensating for shrinkage. By pressure grouting Mr. Jorgensen has been able to force the crest of a dam up stream, thereby demonstrating that the process was something more than merely filling the contraction joints. Fredrik Vogt, Assoc. M. Am. Soc. C. E., has applied for a patent on a multiple grouting system,[31] which aims to equalize the arch stresses, as far as possible.

Mr. Wiley's Conclusion No. 4 reads:

"If the contraction joints should open so as to prevent arch action, water entering the joints would reduce, by its back pressure, the effective head on the dam and compensate for the loss of arch action."

The writer does not understand this. As already stated, there may be no crack, but there may be tension, which would prevent arch action just as effectively as an actual crack. Possibly Mr. Wiley could work out an example in figures to illustrate and amplify his remarks.

Mr. Henny refers to the experience with the Crystal Springs Dam in the San Francisco earthquake and concludes that very severe earthquake shocks may be borne safely by straight gravity dams. Professor Ransome refers to the same dam, and neither calls attention to the pertinent fact that the dam referred to (the San Mateo Dam) has a section about twice that of the ordinary so-called gravity dam, which is designed with no allowance for uplift and is barely stable when the reservoir is full. The San Mateo Dam is 146 ft. high, 41 ft. wide at the crest, and 176 ft. at the base. The St. Francis Dam, which was a typical so-called gravity dam, had a crest width of 16 ft., and, for a height of 146 ft., its base width was 101 ft.[32] The San Mateo Dam has a stability of approximately four times that of most so-called gravity dams. It is evident that any experience with the San Mateo Dam does not apply to gravity dams in general.

---

[28] *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 478.

[29] *Loc. cit.*, Vol. 92 (1928), p. 813.

[30] "Improving Arch Action in Arch Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 316.

[31] "Distribution of Major Stresses in Arch Dams Should Be Uniform Throughout the Structure," *Hydraulic Engineering*, December, 1929, p. 734.

Mr. Henny calls attention to the fact that concrete shrinks in setting, largely due to the cooling action. Transversal shrinkage cracks in dams have been often observed, and Mr. Henny suggests the possibility of shrinkage cracks inside the dam, the planes of which would be more or less parallel to the up-stream and down-stream faces. Such cracks have appeared in a number of multiple-arch buttresses, and the possibility of similar cracks in gravity dams was first suggested to the writer by Dr. Vogt, in 1927, after he had inspected some high gravity dams. Some such cracks were found in a gallery of the Elephant Butte Dam, but these extended only a few inches from the surface. They were, moreover, very irregular, thus suggesting that they were only surface cracks. The writer thinks there is much less danger of such cracks developing in a gravity dam than in multiple-arch buttresses, but Dr. Vogt's suggestion merits careful consideration. So far as the writer knows, no such cracks have been found in gravity dams, and Mr. Henny does not mention any examples from his own experience. He refers to some contraction joints parallel to the axis of the dam as being provided in the design of the San Gabriel Dam. It is the writer's understanding that these joints have been abandoned, and in his opinion such an innovation should not be tried in the highest dam thus far attempted.

The one serious objection to proceeding with higher dams of the gravity type has not been mentioned by either Mr. Wiley or Mr. Henny. This is the question as to the accuracy of the fundamental assumption that the vertical stress distribution across a horizontal plane is linear. There is neither experimental nor theoretical verification of this assumption, which has been borrowed, somewhat uncritically, from the theory of bending of long beams. Whatever evidence of a theoretical and experimental nature there is, it is against the usual assumption. The tests of Messrs. Ottley and Brightmore in 1908, referred to by Mr. Henny, as well as the tests by Messrs. Wilson and Gore, clearly indicate a non-linear stress distribution, as do also recent tests made by Dr. Vogt. Application of the principle of least work to this problem also gives a curved stress distribution and shows tension in the up-stream face, in cases where the ordinary theory gives compression. Even a small amount of tension in the up-stream face is extremely dangerous, in that there is a tendency for this to extend itself, especially if aided by uplift.

---

[32] *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 223.

The writer concludes, therefore, that tests to determine the actual stress distribution in gravity dams are badly needed before the height limit is materially increased.

As to whether a high arch dam is to be preferred to an arched gravity dam of equal height, that, in the writer's opinion, depends upon the site and its general characteristics, and no general rule can be laid down. If there is any question about the foundation being sufficient for a high arch dam, the writer would not consider a gravity dam either, unless it was of extraordinary dimensions, such as the San Mateo Dam. As between a properly designed arch dam and a gravity dam designed as outlined by Mr. Wiley, that is, on the assumption that the convergence of the radii and the influence of uplift are offset by arch action, the writer would invariably prefer the arch dam.

Professor Ransome presents the viewpoint of the geologist. Any careful engineer will always insist upon obtaining competent geological advice and will then make his own investigations and tests and draw his own conclusions, bearing in mind that the responsibility is his. Geology is, relatively speaking, a general science. It is often limited to reasoning on possibilities and probabilities and from effect to cause, which is always a precarious and dangerous proceeding, unless its limitations are well understood. It is the geologist's office to inform the engineer of the general nature of the surroundings, the character of the rock, the probability of earthquake movements and shocks, their probable intensity and direction, etc. It is not the office of the geologist to express himself regarding the stability of a dam. That is a matter to be decided by the engineer after carefully made calculations based upon such experience as he may have at his disposal.

The geologist reporting on dam sites needs some general understanding of what kind of information the engineer wants, and the engineer should bear in mind that geology is general rather than specific. In this way the best co-operation will result between engineers and geologists, and such co-operation is absolutely necessary in order to avoid costly and fatal mistakes.

Allen Hazen,[33] M. Am. Soc. C. E. (by letter).—At a time when more and higher dams are being built and when examples of dams that have failed or partly failed, are accumulating at an unprecedented rate, Mr. Henny's words of caution are most timely. Dams certainly do not follow the law of models, and proportions that stand at one height cannot be taken as an argument for safety in a much higher dam of the same proportions.

In presenting the subject of safety in dam construction to the World's Engineering Congress in Tokyo, Japan, in November, 1929, the writer pointed out that the function of a dam is to absorb the water pressure upon it and carry it to the sides and bottom of the valley and to distribute that pressure, together with that resulting from the weight of the dam, over and through enough of the natural material in place so as not to stress any part of it unduly. The pressures that must be absorbed and transmitted are among the largest that engineering structures of any kind are called upon to carry, extending to hundreds of thousands and even to millions of tons.

---

[33] Cons. Engr. (Hazen, Everett & Pirnie), New York, N. Y. Mr. Hazen died on July 26, 1930.

A strong, light dam may concentrate the pressure on the sides and bottom of the valley beyond the ability of the foundation material to withstand; and it avails nothing to have the dam strong enough if the natural material is not fully competent to hold the resulting stresses.

Mr. Henny does not mention the thought that has been held by some engineers for many years, namely, that a gravity masonry dam of dimensions such that the line of thrust comes at the limit of the middle third, in reality, does not have any proper factor of safety. Many such dams have been built and some of them have failed. It may be true that those that stand depend upon tensile strength in parts of the masonry and resistance to torsion and shear and similar secondary means of resistance for any factor of safety that they may have. Furthermore, it may be that the weight of such a dam is in almost exact balance with the water pressure and is stressed by that water pressure so that if secondary sources of strength are excluded it is stressed almost to the point of rupture.

Nearly all dams in the last analysis are held by friction. It may be friction of masonry on masonry, or masonry on rock, or rock on rock. The friction in all these cases and at all possible lines of slipping must be sufficient to take up the thrust of the water pressure.

There is reason to think that the friction of masonry on some rocks and of the rocks on themselves may be much less than has been commonly supposed—and much less than would be prudently assumed for granite and other hard rocks. There is also reason to think that the frictional resistance of some rocks does not increase in proportion to pressure as is usually assumed.

The sliding of great dams presents many interesting problems which seem to have much in common with such phenomena as the slides at the Panama Canal and other great landslides. There is much uncertainty as to many factors, and increased weight is, in general, the surest road to safety.

H. DE B. PARSONS,[34] M. AM. SOC. C. E. (by letter).—While the four papers, forming the Symposium, contain many valuable suggestions, there are some statements that the writer would like to discuss.

Mr. Wiley lays stress on "the necessity of hard and durable rock foundations" as a rigid limiting condition for masonry dams. Such a condition certainly is much to be desired, but is not always available. A safe masonry dam can be built on "poorer" rock to heights that could be considered high, if given a suitable section. For a thin arch dam, however, "good" rock is essential. It is questionable whether the horizontal arch action in an arched gravity dam, having the same cross-section as a straight gravity dam, would offset the reduction in gravity action due to the convergency of the radii. The amount of horizontal arch action obtained would depend on the length of the radius and on the height of the dam. The water pressure on a gravity dam is considered as carried by the cantilever action. Assuming the base rigidly fixed to the rock, the deformation of the masonry would be a function of the height. If the same dam were arched, the radius would have to be relatively short or the dam very high before the compression of the masonry, due to deformation, would cause an horizontal arch thrust that would be truly effective at the buttress ends. With a long radius, the deformation probably would be taken up by closing vertical contraction joints.

---

[34] Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.

A reading of Mr. Wiley's paper leaves the impression that hydrostatic uplift is not an important element, especially under arched gravity dams. All reported observations under actual dams show the presence of uplift.[35] Uplift would seem to be an actual force, like gravity or reservoir water pressure. Recorded observations show that grouting under the up-stream edge of the base does not eliminate uplift, and the assumption that it will is not founded on fact. It may lessen the uplift, but the reduction is questionable. When grouting is practiced, it would seem safer to use low pressures. The high pressures (say, from 80 to 150 lb. per sq. in.) often used may open the seams in a rock foundation and thereby cause increased uplift.

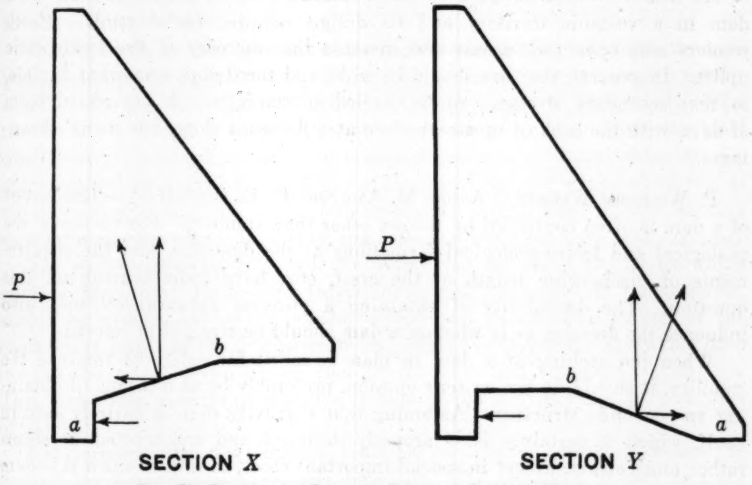


FIG. 7.—EFFECT OF FORM OF BASE OF DAM ON SLIDING.

The conclusion may be true that "there is no uplift * * * in foundation rock which is free from seams, nor in a perfect connection between the dam and the bed-rock", but such perfect conditions are not found at the sites for high dams. It would not be safe to design an important dam or a high one under such assumptions. Uplift under the base of a dam is a natural force, and the shape or material of a structure does not change this law of Nature. If it exists under a straight structure, it would also exist under an arched one.

In Mr. Henny's paper hydrostatic uplift under dams is given recognition. The writer believes that it cannot be entirely "prevented" although it may be reduced. The form of the dam base may have an effect on sliding due to

---

[35] "Hydrostatic Uplift Under Masonry Dams," by H. de B. Parsons, M. Am. Soc. C. E., *Paper No. 313*, World Eng. Congress, Tokyo, Japan, November, 1929.

hydrostatic pressure. In Fig. 7, two sections for gravity dams are shown that differ in their base planes. Hydrostatic head acts normally to the surface. With Section *X*, the head acting against the cut-off, *a*, and the horizontal component acting on the slope between *a* and *b*, would be opposed to the reservoir pressure, *P*, and, therefore, would diminish the tendency to slide. With Section *Y*, the horizontal component of the hydrostatic head acting on the slope between *a* and *b*, would be in harmony with the reservoir pressure, *P*, and would thereby increase the tendency to slide.

Deflection of the masonry, acting as a cantilever, affects the stresses at the up-stream edge of the base. At this edge, uplift is usually at a maximum. Therefore, the stress across the base joint, between the rock and the masonry (or concrete), may be changed from compression to tension, and the intimate contact between masonry and rock may be weakened or the rigid connection lost.

As clearly pointed out in Professor Ransome's paper, the foundation for a dam in a volcanic territory and its design require special study. Earth tremors may open rock seams and increase the intensity of the hydrostatic uplift. In general, the base should be wide, and the design somewhat flexible, so that secondary stresses can be carried successfully. A buttressed form of dam, with the back or up-stream face at a flattened slope, has many advantages.

P. WILHELM WERNER,[36] ASSOC. M. AM. SOC. C. E. (by letter).—The layout of a dam is often controlled by factors other than stability. For instance, the geological and hydro-geological formations at the dam site, and the requirements of discharging length of the crest, etc., have some bearing on this question. The desirability of obtaining a pleasing appearance[37] may also influence the decision as to whether a dam should be straight or curved.

When the arching of a dam in plan is called for solely to improve the stability, it should, in the writer's opinion, preferably be as a means of obtaining an economic structure. Assuming that a gravity dam is entirely safe in itself, which it certainly is if properly designed and constructed, it seems rather unnecessary, except in special important cases, to impose upon this dam an additional condition involving expenses with a view to increasing the factor of safety. This is especially true, if the conditions are such as to give the arch action only a more or less hypothetical value, until the cantilever has been broken.

Obviously, there is a continuous transition between the pure arch and the gravity section. The question as to whether, under certain conditions, a division of the load takes place between the arch and the cantilever need not be discussed. The writer believes that this is often more or less a matter of computation, and lately it has been so treated in many instances.[38] It should be carefully noted, however, that governing factors such as shrinkage, etc., are rather difficult to compute exactly, necessitating an ample allowance on the safe side. Any uncertainty as to the assumptions adopted for a theoretical investigation of this nature should be carefully considered in the light of actual experience on dams already constructed, etc.

---

[36] A. B. Vattenbyggnadsbyran, Stockholm, Sweden.

[37] "The Design and Construction of Dams," by Edward Wegmann, Eighth Edition, N. Y. and Lond., 1927, p. 162.

[38] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1191.

The writer would not consider it safe to state that an arch of any required radius is preferable to a straight dam. It is undoubtedly true, however, that an arched dam has a larger factor of safety against ultimate failure than a straight dam of the same cross-section. The arched dam may stand by arch action, even if the cantilever has been broken, provided the abutments are capable of taking the arch thrust. In this there is, perhaps, a nucleus of a rational use of the gravity section, considering the possibility of making less provision for uplift. When doing this, however, engineers should realize clearly that they may have sacrificed some of the factor of safety against failure.

Back pressure on contraction joints or in shrinkage cracks would undoubtedly reduce the effective head on an arched dam. The question is, however, whether it is advisable, without special arrangements, to count upon it for the stability of the dam.

It might happen that full back pressure exists in one joint, but that the next joint is partly or wholly drained. This would evidently involve an unfavorable condition for the block that is subjected to the side pressure. It is questionable, therefore, whether it is consistent with a safe design to count upon the ideal conditions of full back pressure on contraction joints or in shrinkage cracks, unless special measures are taken to seal the joints and the cracks effectively at the down-stream end. It would be exceedingly interesting to know whether such measures have been applied in any case, and how they have been executed.

The writer agrees entirely with Mr. Wiley that the use of any device which offers a very short distance of travel for percolating water in the concrete, should be avoided as far as possible. However, an effective sealing of the contraction joints at the down-stream end tends to concentrate the whole drop in pressure to the down-stream face of the dam and especially to the toe. This may cause deterioration and corrosion of the concrete in that vital part of the dam.

Furthermore, the down-stream seals in contraction joints may lead to unexpected consequences as regards the uplift. The admission of back pressure in the joints should inevitably involve a considerable increase in the allowance for uplift in and under the dam.

If the contraction joints are sealed at the up-stream face, it does not seem reasonable to assume full back pressure on the joints. Apparently, this question has a great bearing on the design. Therefore, it would be of the greatest interest and of the utmost value to have a number of completed dams visited by a committee of experts so as to determine whether or not reliance can be placed on such back pressure. In the writer's opinion it does not seem consistent with a proper design to accept an unsafe assumption on the ground that it sometimes happens to be correct.

An arched gravity dam without support from the arch and with no back pressure on the joints would have a smaller factor of safety against partial failure than a straight dam. The writer has also shown[89] (without advocating the use of the design, however) that a curved dam with a convex down-stream face under certain conditions may be more stable than a straight dam of the same cross-section. This is not a fallacy, because there is always the possibility of draining the joints, so that back pressure shall not occur. If it existed in this case, it would increase the effective head on the dam.

---

[89] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 784.

Uplift In and Under Dams.—In a homogeneous mass consisting, for instance, of cement or lime, there is probably no uplift, because the forces in each void (as explained by Mr. Wiley) are opposed and cancelled by equal forces in the voids below. If this is correct, however, there must consequently be uplift at a point of perfect contact between the masonry and a rock that is free from seams, because in a tight rock there are no voids below.

As a matter of fact, there must always exist an unbalanced pressure in planes between strata of unequal porosity. Such planes are constituted by the foundation and, to a certain extent, by construction joints or other places of non-homogeneity in the dam.

Uplift conditions do not seem to be the same in concrete as, for instance, in cement or lime, Professor P. Fillunger has shown[40] theoretically that uplift exists in a mass of concrete due to the content of sand and stone. He has also derived formulas for the computation of the uplift in a dam subjected to percolating water.

C. A. P. TURNER,[41] M. AM. SOC. C. E. (by letter).—The tendency to build dams of ever-increasing height is statistically shown by Mr. Henny. In general, concrete rather than masonry is best fitted for such structures because of lower cost; and reinforced concrete is preferable to plain concrete. A dam 400 or 500 ft. high presents a structural problem of the first magnitude, requiring for its most economic but safe design ingenuity of a high order, checked by the rigid application of exact elastic theory to the resistance of the materials involved.

Foundations.—For the high gravity type of dam the abnormal quantity of concrete required to follow Rankine's view of stability has been reduced by the hydraulic engineer with such increase of toe pressure that it equals or exceeds the shearing strength of softer rock and only a solid ledge of hard rock, such as granite, trap, dolomite, etc., is fit to sustain it.

The customary margin of safety of 100 or 125% in engineering structures has, in the design of high gravity dams, been reduced by the hydraulic engineer to so small an amount that dams with base widths of 66% and 85%[42] of the height, as compared with Rankine's 100%, are now being discussed. Why not revert to the structural engineer's idea of a liberal 100% margin of safety? Why not adopt a base of one and one-third or one and one-half times the height so that safety against overturning would be apparent without computation or argument? Why not incline the up-stream face of the straight dam so that the resultant water pressure will push the dam down on its base, increasing its stability and shearing resistance to sliding, instead of following the unscientific practice of building a vertical up-stream face with a resultant pressure tending to overturn the dam, to raise the heel, and to crush the toe, thus illogically reducing, instead of increasing, resistance to sliding?

---

[40] "Auftrieb und Unterdruck in Talsperren," by Prof. P. Fillunger, *Die Wasserwirtschaft*, No. 20–21, Vienna, 1929.

[41] Cons. Engr., Minneapolis, Minn.

[42] *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, pp. 166–174.

The carpenter millwright of years ago built timber dams applying this reasoning to his construction. The multiple-arch type of dam in which buttress walls at intervals of about 30 ft. support arched concrete plates is one resultant of this line of thought, and such structures have permitted the economic development of irrigation projects that would not have been commercially feasible with the more cumbersome gravity type.

The objection urged by some against the multiple-arch type is that if a single abutment should slip or be undermined then arch after arch would fail consecutively. A further objection is urged against the thinness of the concrete plates, but this objection is based upon disintegration of porous and inferior concrete.

From the construction standpoint buttress walls in the multiple-arch dam cannot carry the great weight incident to the high dam as economically as hooped and longitudinally reinforced columns; nor can the arches be centered from abutment to abutment as inexpensively as a flat slab from column to column.

The writer's opinion as to the most economical type for the long straight high dam is a flat slab or an inclined plate design with a relatively flat up-stream slope, or batter. This slab can be supported by reinforced columns, or struts, battered to coincide with the direction of the resultant of the water pressure and vertical load. With an inclination of the plate at an angle of 40° to the horizontal the base is more than one and one-half times the height.

The quantity of concrete required for such a structure is approximately one-third that required for mass concrete of the gravity type; and when the surface rock can safely carry 20 tons per sq. ft., the quantity of concrete required would not be one-fourth as great. Offsetting the saving in part is the reinforcement and higher grade of concrete used. The margin of safety will be much more than that of the ordinary engineering structures, in order to guard against the damage and loss of life that follows failure.

Curved Dams.—This type may be divided into two classes, those in U-shaped canyons and those in V-shaped canyons. In the first class are short high dams between rock walls that are nearly vertical and in which the resistance is predominantly arch action when the height is three to four times the length of the crest. Computation on the assumption that the entire resistance is arch action will lead to a safe structure without excessive waste. On the other hand, with a V-opening and a length of crest two and one-half times the height, the resistance is predominantly a twisting resistance. The down-stream component of the normal water pressure twists the curved plate because the base cannot deflect elastically down stream so as to permit arch resistance. The cross-stream component of the normal pressure gives rise to secondary arch action in which the cross-stream pressure is comparable to the abutment thrust of the true arch. That the resistance is predominantly a twisting resistance was demonstrated in the experiments on the Stevenson Creek Dam.[5] The deflection of the end quarters of the crest was up stream against pressure.

No deflection occurred at the quarter-point, but the central half of the crest deflected down stream.

In masonry structures the old craftsman broke joints in his natural stone masonry, thereby attaining increased resistance approaching that of a monolithic mass. In the artificial stone of the concrete dam this practice has not been followed because of the exaggerated idea of temperature stress presented in engineering handbooks and taught in works on strength of materials.

From observation it appears that the steel stress necessary to restrain the contraction of concrete through a wide variation of temperature is less than one-tenth that computed according to handbook rules. At Moose Jaw, Sask., Canada, a flat slab structure with expansion joints between 300 and 400-ft. centers was inadvertently built with rails not cut where joints were made in the slab. These two rails (65 lb. per yd. in section) restrained the contraction of the slab of 9 000 sq. in. cross-section under a drop of 60° Fahr. After the rails were cut, the joint opened from ¾ to ⅞ in. with a loud report. Ordinary computation would indicate a stress in the rails six times their yield point value and twelve times the strength of the fastenings if the contraction of the concrete were assumed to allow 2 000 000 lb. for the modulus of the concrete, which tested more than 4 000 lb. per sq. in. Considering the steel and the tension which would be required to elongate the steel ¾ in. in that length and dividing this by the area of the concrete, the tension in the concrete must have been so small that a review of the basis of ordinary computation is in order.

This method assumes without experimental verification that an elastic material under longitudinal strain expands the same amount laterally as if the longitudinal strain were lacking. Investigation of the thermo-elastic properties of matter by Kelvin indicates roughly that the shift of internal energy of vibration from kinetic to potential, or *vice versa,* represents an amount of energy twice the external work of deformation. Hence, it is logical to conclude that the temperature stress is one-half or less than one-half in the homogeneous material than that which is ordinarily computed by textbook rules. In dealing with the composite mass of concrete and metal the tension in the steel opposes the contraction of the concrete under a drop in temperature, and the steel transmits this tension to the concrete through shear, resulting in a rotation of the direction of the principal tensions 45° to the axis of the steel. It is reduced, therefore, relatively to one-half the magnitude of the restraining tension of the steel and distributed through the section in such a way that, were there diagonal reinforcements in addition to longitudinal, it would be further reduced.

As previously noted, when the two rails were sawed nearly through, the nicked section broke with a bang. The cross-sectional area thus broken, multiplied by the ultimate strength of the material when so nicked, furnishes a definite measure of the contraction stress on the 9 000 sq. in. of cross-section under a drop of temperature of approximately 100° Fahr. The total stress distributed across the cross-section would be, on this basis, less than 90 lb. per sq. in. A contraction force distributed between the longitudinal and the diagonal steel reinforcements and the concrete itself leaves a tension of less than 40 lb. per sq. in. in the concrete matrix developed by the drop in temperature of 100° Fahr., which is an amount too small either to crack or to check the concrete.

This fact accounts for the lack of difficulty which the writer has had in building flat plate dams 3 000 ft. in length without expansion joints, and bridge floors 500 to 600 ft. long without any provision whatever, under temperatures ranging from 100° above to 50° below zero. It indicates the practicability of the monolithic concrete dam when properly reinforced, so that the full resistance of a monolithic mass may be secured without serious checking or cracking of the concrete. The advantage of the well-reinforced monolith lies in its toughness, as illustrated by the example of the failure of a water tank.[43]

Bridge engineers a generation ago discarded the use of plain concrete because reinforced concrete presents greater toughness and dependability and because a smaller body of reinforced concrete may be used more economically. This practice of the bridge designer, which is the result of extended experience, may well be followed in the design of concrete dams for greater safety and dependability.

Unscientific computation of the maximum tension in the concrete has been presented repeatedly in Society publications, based upon the futile effort to apply the ordinary or approximate theory of flexure (suitable only for determining the maximum fiber stress in a long cantilever) to a short section of a dam having a length only a fraction greater than its thickness. Exact theory which takes into consideration shear strain is essential to practical accuracy and rational conclusions.

Resistance to external shear force is found (dependent upon the relative thickness to span length) to be several times greater per unit of cross-section with continuous and restrained beams and slabs than by the erroneous assumptions on which the complicated ordinary theory of flexure is based.

Although the correct equations for the statical equilibrium of moments was taught sixty years ago,[44] these principles have been so little understood or have been so disregarded by the majority of present-day engineers that computation of curved dams and flat plates supported upon columns or beams differs widely from the simple exact solutions resulting from their rigid application.

The writer's views may be summarized as follows:

1.—For the high dam reinforced concrete rather than plain concrete should be used from the standpoint of economy and safety.

2.—The inclined plate has an advantage in reducing the pressure on the base to approximately one-third that of the gravity type and of reversing the distribution of maximum pressure, so that the maximum pressure occurs at the heel and the minimum pressure at the toe, thus insuring stability.

---

[43] *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 170.

[44] "Natural Philosophy," by Thomson and Tait.

3.—Under settlement, the water pressure with the inclined-plate dam forces the column down upon its bed, and, although the slab may be cracked because of such settlement, it will still carry the load because the twisting resistance of the plate is not destroyed by checking from the top to the bottom.

4.—For the long high dam, the inclined plate is the most economical type, and its stability against overturning is apparent without computation or argument when the base is one and one-half times the height. Such a liberal base adds to the resistance against sliding, and the water pressure itself tends to increase this stability instead of reducing it as in the gravity type.

5.—For the relatively short dam on a rock base in a V-formation the curved dam may cost less than the plate structure; but if the curved dam is built the surfaces should be substantially cylindrical with flatter curvature at the ends and the sharpest curvature in the middle half. Such a dam with a length of crest two and one-half times the height should be computed for its predominant torsional resistance instead of as an arch. It should be built as a monolith and reinforced for temperature stresses.

6.—The curved dam is suitable only where the base presents a ledge of hard solid rock.

7.—The reinforced plate is suitable to dams far higher than any yet proposed. With column spacing of 30 ft. for a 500-ft. dam, the diameter of the lower columns will be approximately three-eighths to one-half their distance center to center; and the bracket heads will furnish a complete sub-arch from column to column where the pressure is greatest.

Were the dam designed with no greater margin of safety than the ordinary multiple-arch structure, the reinforcement could be treated as temperature ties for the thicker panels. Conservative practice, however, will dictate the computation of the steel on the exact theory of flexure, so that the margin of safety will be greater than that in bridge design.

In view of the enormous damage and loss of life that attends the failure of a high dam, why not make the structure so tough that it would be difficult to crack up with dynamite, instead of so fragile that a little leakage and undercutting is sufficient to cause it to crumble and fall apart, letting loose a deluge as in the case of the St. Francis Dam failure? There is no excuse for a high dam that does not embody such toughness and safety, since the cost is much less than that of the gravity type and the work may be executed with greater facility and rapidity.

F. W. HANNA,[45] M. AM. SOC. C. E. (by letter).—The Symposium on High Dams, as presented by the authors, is fundamental rather than exhaustive. It forms, therefore, basic data for elaborating more complete information from which to develop principles of dam design. In this respect it is a useful contribution.

The question of what is a high dam has not been answered in the Symposium; and, indeed, it cannot at this time be subject to answer in a single figure for any type of dam or for dams as a whole. Table 1 gives the highest dams constructed or under construction, at the present time (1930).

---

[45] Chf. Engr. and Gen. Mgr., East Bay Municipal Utility Dist., Oakland, Calif.

These data represent the present extremes of practice, and indicate to the writer somewhat of the relative future heights of the various types of dams. It now appears that the arch and the gravity types are well in the lead in this respect. The second highest gravity dam of any type in existence is the Pardee (Lancha Plana) Dam, 358 ft. from base to top, and the second highest arch dam is the Pacoima Dam, 375 ft. from base to top. The San Gabriel and Boulder Canyon Dams, both of the masonry gravity type, have been projected for still greater heights, the former for 500 ft. and the latter for 700 ft. These dams indicate the possible future trend of gravity dam heights. However, the San Gabriel Dam has been abandoned on account of a defective foundation, and the Boulder Canyon Dam has not been started and may be changed in design before it is built.

TABLE 1.—Highest Dams of Various Types

Name.	Type.	Height above foundation, in feet.	Place.
Rodriquez	Ambursen	225	Mexico.
Cobble Mountain	Earth	245	Massachusetts.
Lake Pleasant	Multiple-arch	256	Arizona.
Salt Springs	Rock-fill	300	California.
Kensico	Straight gravity	307	New York.
Diablo	Arch	400	Washington.
Owyhee	Arched gravity	405	Idaho.

Theoretically, both earth and rock-fill dams on suitable foundations may, by proper dimensioning, be made very high; but their lack of resistance to disintegration on overtopping, their liability to destruction from leakage, and their great bulk for their height, due to increasing side slopes with increasing heights, will probably never permit them to rival in height the arch or gravity masonry dams.

The buttress types are generally built with comparatively thin sections requiring reinforcing steel for stability in design, which may limit their lives too much for high, expensive, important dams. Thin sections are also poor resisters of seepage, frost, and earthquakes. Consequently, the buttress types are not likely to be the rival high dams of the future. This, of course, does not limit their usefulness for relatively low dams.

Through the elimination of the earth, rock-fill, and buttress types for the various reasons cited, the high dams of the future are likely to be, as they are now, either the gravity or single arch type. As to which of these will be the final pinnacle of height depends on the natural formation of canyons. The gravity type will no doubt lead for wide, and the arch type for narrow, canyons. The limiting heights of these dams on hard impervious rock foundations will be determined by the allowable working stresses in good concrete; and the dam that can come within these stress limits at least cost will ultimately be chosen. In the very high dams of these two types, stress limitations now come into play; but it is certain that stress allowances will be greatly increased with better concrete mixtures and manufacture; perhaps they may be doubled in the near future.

The arched gravity dam is superior to the straight gravity dam only in cases where the arch length is not too great, where the canyon walls will resist pressure, and where the foundation contours make it the cheaper. It is evident that no advantage results from arching the gravity type dam when the canyon side-walls are not suited to taking arch thrust. In very wide canyons no benefit is derived from arching the gravity dam because excessive deflection or buckling of the arch slice may result if the slenderness ratio is too great. When the height from foundation to crest decreases from the center toward the abutments the greater deflections of the longer cantilevers throw torsional stresses, sliding tendencies, and overturning moments into the shorter cantilevers toward the ends of the dam. The arch resistance helps the shorter cantilevers to absorb these loads. In the case of a straight gravity dam these loads can be resisted only by increased sections. That arch action actually takes place in an arched gravity dam is clear from the effects of cantilever deflection, due to water load and soaking of the up-stream face. In the computation of the stresses in the arched gravity type of dam by the usual method of taking a vertical slice of one unit width, at the up-stream face, the parallel plane theory should generally be used because such a slice between radial planes cannot slide or rotate out of place independently and this method of calculation does not give credit to the full weight of the material in the dam.[46]

For narrow canyons with sound rock side-walls, the arch dam is superior to the gravity dam on account of economy of materials and greater factor of safety. That the volume of the former is much less than that of the latter is evident. The factor of safety of a gravity dam against sliding may generally be only a little more than 1, and seldom more than 2, whereas that of a well-designed arch dam of the type tentatively suggested by the writer[47] for the Pardee Dam is 5 or 6, or more. In addition to the usual factors of safety provided for shear and compression stresses, such an arch dam has unknown additional factors of safety due to torsional resistance, to transference of stresses to the foundation by the shortest route, and to a limited degree cantilever beam action. That torsional stresses exist in arch dams has been pointed out by the writer.[48] These stresses and the corresponding resistance may be computed in each arch slice by considering the resistance to twisting of the voussoirs in it as the arch deforms under its load. The effect of arching by the shortest route to the abutments is very difficult to ascertain, but that it often exists to a considerable degree is certain. Neither the trial-load method nor the single-center cantilever method of computing the cantilever beam resistance is very reliable because of incorrect basic assumptions. Moreover, the dry down-stream face and the wet up-stream face may produce deflections in the cantilevers that bring arch action into play and relieve the cantilevers of their loads. The fear of tension in the up-stream face of an arch dam designed as suggested is not well-founded, as cracks produced by such tension only extend into the dam a short distance and the uplift is prevented partly, if not altogether, by water soaking. This is not true of tension in the up-stream face of, and uplift in, a straight gravity dam, where increased overturning resistance is called into play. The arch dam is now, and is bound to be, a strong rival in the future for the position of maximum height in masonry dams.

---

[46] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), pp. 799–808.

[47] *Engineering News-Record*, March 15, 1928; *Western Construction News*, May 25, and June 25, 1928; and February 10, 1929.

[48] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 1246–1250.

Pressure grouting has been used with splendid success, both in the foundations and in contraction joints of some of the high dams now constructed, and will be used quite probably in all high masonry dams in the future. It is the writer's opinion that no high dam either of the arch or gravity type should ever be constructed without a complete grout curtain under the heel, put in under the highest practicable pressure without disturbing the rock strata of the foundation or the contact of the dam with the foundation rock.

It has also become quite general practice to construct both arch and gravity dams with vertical construction or contraction joints at intervals perpendicular to the up-stream face of the dam and sealed with metal water-stops. These joints should also be grouted under strong pressure to insure water-tightness in all high masonry dams and also arch action in the arched gravity dam and the arch dam. There is no difficulty in securing thorough distribution of the grout in the joints when they are properly designed.[49]

The protection against uplift in a high masonry dam by the addition of a drainage system to the grout curtain already discussed, seems to be gaining ground. The drainage system for a gravity dam should be made to consist of a cut-off trench near the up-stream toe, connected by tile pipe with a series of wells arranged to discharge into an inspection gallery of such proportions as to permit drilling. For an arch dam, drainage holes should be drilled from the down-stream face of the dam into the foundation to points near the up-stream face, thus relieving pressure.

It is also being appreciated that in high gravity dams of considerable thickness cracks are liable to occur parallel to the faces of the dam and it has been proposed to place contraction joints in such dams to control these shrinkage cracks. Whether this can be done without special treatment to care for the stresses is not yet certain because continuity of stresses from heel to toe and top to base of these dams requires continuity of resisting material. In the case of a smooth contraction joint parallel to the down-stream slope of the dam, the slab down stream from the joint may, if the joint opens, crack near its middle under its load, and fall against the adjoining concrete; or it may act as a column or strut tending to overturn the top of the dam up stream, thus producing tension on the horizontal planes.

H. F. DUNHAM,[50] M. AM. SOC. C. E. (by letter).—There is a measure of assurance in Mr. Wiley's discussion of the effects of contraction in the curved gravity type that must be gratefully regarded by those who design high dams. The same confidence shows in the reference to uplift. If up-stream, vertical contraction cracks are not a serious weakness, and if uplift can be greatly lessened through the absence of smooth horizontal surfaces, two features in construction will seem less formidable.

---

[49] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), pp. 799–809; *Engineering News-Record*, March 21, 1929.

[50] Civ. and Hydr. Engr., New York, N. Y.

While the writer's approach to the question of uplift as stated in an earlier discussion[51] differed from that of Mr. Wiley, the result was not at variance with his conclusions. There may be noted, too, the similarity with respect to saturation of deep quarry rock and the concrete in a remaining part of the St. Francis Dam.

To avoid metal seals, the writer, in 1928, proposed the use of heating units (electrical, steam, or hot water), to be placed near each face of a dam, but not too close together. The pipes or pipe lines should be easily drained; they should be free to expand; and all should be connected to one source of heat. Used in, or prior to, cold weather the heat would be distributed and stored to offer a barrier to frost by increasing radiation. The involved principle of heat transference would be similar in some respects to the charge of a storage battery from a small unit. The slow increase in heat would be available when needed. The expense may appear favorable if set against the cost of extra masonry useful, in part, as a blanket to protect that section of a dam above the normal up-stream water elevation.

LARS R. JORGENSEN,[52] M. AM. SOC. C. E. (by letter).—Much discussion has taken place recently as to the exact action of a gravity dam arched in plan. Mr. Wiley favors an arched gravity dam and evidently believes that under all conditions such a dam would have the resistance of the arch in addition to its weight. His conclusion that "the contraction joints of an arched gravity dam do not prevent arch action" needs considerable explanation. Most large arched gravity dams built—such as the Exchequer Dam in California—have open contraction joints in late winter. Therefore, it is physically impossible for arch action to be present in the winter when this dam supports full water pressure.

Such a dam, therefore, can not have a larger factor of safety under all conditions than a straight dam of the same section; most likely it has a smaller factor. If it had a larger factor of safety the St. Francis Dam would probably still be standing because it might not then have developed tension in the up-stream face. The Exchequer Dam has considerable tension in the up-stream face in the winter when it can receive no help from arch action. It is built on a good foundation and, therefore, apparently can take care of this tension, but it is not correct under such conditions to call it a gravity dam.

There are enough uncertainties in connection with the design assumptions and the actions of gravity dams to demand a base width of not less than 75% of the height for medium high dams and considerably more for the higher dams. There is no satisfactory substitute for weight in a gravity dam.

Mr. Wiley makes the statement that "at present the arched gravity type of masonry dam is slightly in the lead with respect to height". This is not in accordance with facts. The three highest dams, holding back the greatest depth of unsupported water load are arch dams, such as the Drac River Dam in France, 430 ft., the Pacoima Dam, 365 ft., and the Diablo Canyon Dam, 363 ft. There is no reason why this type should not or could not be built at least as high as any gravity dam on sites where it would fit, and be safer and cheaper. The safety of gravity dams, in comparison with other types, especially single arch types, has been greatly over-rated in the past. For very high arch dams it would be good engineering to use higher stresses than those now generally acceptable, say, up to a maximum compression of 1 000 lb. per sq. in., and to ensure that no concrete would go into the dam of less ultimate crushing strength than 4 500 lb. per sq. in.

---

[51] *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 246.

[52] Cons. Hydro-Elec. Engr., San Francisco, Calif.

This would not be at all difficult. Such a dam would have a factor of safety in excess of that of any high gravity dam that would be commercially feasible to build. Besides, in the majority of cases the arch dam would be much cheaper.

William P. Creager,[53] M. Am. Soc. C. E. (by letter).—With particular emphasis on the outlook for the future, the writer has often wondered if the theory of design, as applied to existing dams, is adequate for very high dams. The tendency is for higher dams, and it is now proposed to build a dam 700 ft. high, or nearly twice as high as the highest dam in existence. The Quebec Bridge failure taught a lesson in regard to the utilization of theories for large members or structures which have been proved adequate only for small members. Has this lesson been well learned by the designers of dams?

As a word of caution, the writer desires to call attention particularly to two features of design which, in his opinion, are in doubt for high masonry gravity dams.

The force required to move a block of stone or concrete on a like surface has been determined by tests to lie between 55 and 70% of its normal weight. Actual coefficients of friction in a large number of dams have been adopted within this range. Therefore, without consideration of the shearing resistance due to roughening of the foundations and adhesion of the concrete to the rock, a factor of safety of only 1.0 has been provided. It has been assumed, however, that this additional resistance to movement, termed "shearing resistance" in this discussion, is enough to provide sufficient margin of safety. Is this margin sufficient for very high dams? Assuming homologous triangular sections, the total shearing resistance of the foundation varies as the height of the dam, whereas the sliding force of the impounded water varies as the square of the height. Therefore, the shearing resistance is seven times as effective in a 100-ft. dam as it is in a 700-ft. dam. In other words, a shearing resistance of only about 30 lb. per sq. in. is necessary for a factor of safety of 2.0 in a 100-ft. dam; but, in a 700-ft. dam, 7 times 30, or 210 lb. per sq. in., is necessary. Actually, a slightly less shearing resistance than 210 lb. would be necessary owing to the greater spreading of the base of a 700-ft. dam to limit compressive stresses.

In the case of resistance to overturning, present theory requires that the resultant for reservoir full must lie within the middle third, because otherwise there will be tension in the base with the resulting tendency to open cracks and admit full head-water uplift and progressive failure by overturning or sliding. The elastic theory varies this rule somewhat, but is outside the present argument.

---

[53] Vice-Pres. and Chf. Engr., Northern New York Utilities, Inc., and The Power Corporation of New York, Watertown, N. Y.

As many dams have been designed with the resultant close to the extremity of the middle third, it must be considered, therefore, that tension adds the necessary margin of safety. As the total resisting moment of this tension varies as the square of the height of the dam and the overturning moment as the cube of the height, the resisting moment of tension in the base is seven times as effective in a 100-ft. dam as in a 700-ft. dam.

If a dam, with the resultant at the down-stream extremity of the middle third, is assumed to have a factor of safety of 1.0, neglecting adhesion of the dam to the foundation, it requires a tensile strength of about 100 lb. per sq. in. in a 100-ft. dam and 700 lb. in a 700-ft. dam, to provide a factor of safety of 2. As in the previous case, the required tensile strength would be somewhat less than 700 lb., due to greater spreading of the base in a 700-ft. dam.

The foregoing is not intended to be an exact analysis of the subject, but is presented only to indicate that, under the generally accepted practice in the design of dams, a very large dam has a much lower factor of safety than a small one. It seems to the writer that engineers are treading on dangerous ground if they do not apply to the design of very large dams a careful consideration of those factors which, in small dams, are neglected except with the thought that they provide an additional margin of safety.

L. F. Harza,[54] M. Am. Soc. C. E. (by letter).—The gravity concrete dam, whether straight or arched in plan, probably now commands the confidence of the majority of engineers to a higher degree than other types of high dams. It is open to serious doubt whether this confidence is fully justified, and whether existing rules of design can be safely extended to much greater heights without far more knowledge of behavior than is now available.

The factor of safety generally used is much less than factors used in other engineering structures, and is not sufficient to inspire confidence when one considers the many uncertainties, such as possible internal longitudinal cracking; possible tension at the heel; uncertainty as to amount of uplift along the bottom and horizontal construction joints; effect of internal pressure in the concrete; and the fact of reduction in factor of safety against sliding as the height of the dam increases while maintaining the same factor against overturning. The last item has been discussed elsewhere.[55]

The principle of keeping the line of pressure within the middle third of the base, assumes a straight line distribution of pressure from heel to toe. This is an assumption of very questionable accuracy, in a structure having the base width of a high dam, with interior temperatures differing from those at the surface, and with a somewhat flexible toe carrying the heaviest load to a foundation of perhaps not uniform resistance. This assumption ignores several uncertainties that have been pointed out by other writers. For example, it has not been possible, or at least practical, in other concrete structures to build them monolithic for a length equal to the thickness of a high concrete dam without contraction cracks, and such cracks, if approximately vertical, would reduce the effective base width of the dam. Contraction cracks might originate from shrinkage due to overheating during construction, from drying out, from seasonal temperature changes, and from stress.

---

[54] Cons. Engr.; Pres., Harza Eng. Co., Chicago, Ill.

[55] See p. 197.

In the writer's opinion, the middle-third theory is so superficial a principle that confidence in gravity dams can be based principally on reasonably satisfactory past behavior, rather than upon theory. The result is to place rules of design on inductive rather than deductive bases, and to make the design of gravity dams an experimental art, rather than a science. In fact, engineers are not justified in referring to the middle-third method as other than a "rule-of-thumb" procedure, the best support for which is not the mathematical basis, but rather the fact that few failures have occurred.

Current practice in gravity dam design is none too safe, and, in fact, a grave responsibility rests upon the engineer who goes much beyond existing precedent in the height of gravity dams without a far greater knowledge of their internal behavior, and much better control of temperatures and uniformity of concrete, than is now possible.

However, the future is promising because it would seem that engineers are now beginning to learn something about gravity concrete dams. Intensive thought and research is now beginning to focus on this problem, so that the hope may be entertained that engineers are entering an era of research and of scientific study of dam design and the equally important factor of field control. It is probable that, as a result, the old methods will be largely discarded in favor of an analysis involving other than straight line distribution of pressure from heel to toe and other refinements.

There are possibilities of artificially controlling the placing temperatures and the curing temperatures; and several other precautions may be observed, which may assist in building dams that will perform more nearly according to the so-called theory.

In the field of power engineering there has been an ever-decreasing cost of steam energy coincident with an ever-increasing cost of hydro-electric energy; this fact is forcing upon the hydro-electric engineer the exercise of the utmost economy in the design of dams, which form such a large proportion of the cost of a hydro-electric project. Hence, the need for consideration of the merit of earth-fill dams, whether by the dry or the hydraulic method, combined earth and rock-fill dams, rock-fill dams, arch dams, multiple-arch dams, or other types, which may save an initial cost under certain favorable conditions.

An earth dam, well built and constructed under equally good conditions of foundation and available materials, undoubtedly compares favorably in permanency and safety with a concrete dam, when once it is successfully completed. In fact, when once completed, the earth-fill dam is subject to much less in the way of uncertainties than the writer has pointed out in connection with the gravity concrete dam; and it has given as good an account of itself in actual service.

Earth-fill or other loose-fill dams have a well recognized advantage when constructed on soft material or defective rock foundations, because they can usually be accommodated to these conditions, whereas a concrete dam would require very deep excavation, pile, or caisson supports, and often could not be built at all.

A loose-fill dam requires the protection of an ample spillway, and is, therefore, best adapted to situations where a natural ridge is available for large spillway capacity at minimum cost.

The principal uncertainty with an earth or other loose-fill dam is during the process of construction in connection with diversion of the river. Few, if any, earth dams are built, or could be built economically, if provision must be made for diversion during any stage of construction of the maximum flood which might occur on the stream in a long period of years. Construction schedules are, therefore, more important in this case than for concrete dams, and must be very carefully studied and rigidly adhered to in order to permit the work to reach a stage, during a low-water season, which will insure sufficient head to divert the river through culvert or tunnel when the flood season arrives.

Earth dams may be said to be most ideally adaptable when of small volume, so that they can be built between flood seasons, thus avoiding expensive diversion work. They are well suited for small drainage areas where ample diversion provisions would not involve excessive cost. With equivalent dam site topography and foundation conditions, an earth dam would cost much more on a large stream than on a small one, because of the greater cost of diversion; or otherwise a greater hazard would be involved in its construction, if provision was not made for equally safe diversion. The same may be said of a dam of any loose material, such as a combination earth and rock-fill or a rock-fill dam.

In the case of the proposed Boulder Canyon Dam, however, the same problem of diversion is involved, because of the depth of excavation required to reach satisfactory foundation, and this excavating operation must be protected from overflow throughout one or more flood seasons. When a concrete dam is once brought up to river-bed level, however, the hazard of flood during construction is less than with dams of loose material, unless ample diversion capacity is provided for the latter.

A. FLORIS,[56] ESQ. (by letter).—In his paper Mr. Wiley deals briefly with the relative merits of the various types of high dams. In doing so he arrives at certain conclusions which are at variance with the views commonly held by hydraulic engineers. The present discussion will be confined merely to the action of water in the pores of gravity dams.

The problem of water pressure within a gravity dam has been attacked successfully by Professor Paul Fillunger.[57] His treatment of the subject is based on a rigorous mathematical analysis the results of which agree substantially with the tests. His findings are contrary to the opinions advanced by Mr. Wiley, so that it is worth while to compare them.

---

[56] Civ. Engr., Los Angeles, Calif.

[57] "Der Auftrieb in Talsperren," von Dr.-Ing. Paul Fillunger, *Oesterreichische Wochenschrift für den öffentlichen Baudienst,* 1913, Nos. 31, 32, 33 and 34; "Neuere Grundlagen für die statische Berechnung von Talsperren," von Dr.-Ing. Paul Fillunger, *Zeitschrift* des Oesterreichischen Ingenieur- und Architekten-Vereins, 1914, No. 23; also, reprint.

Mr. Wiley reasons that uplift can be produced either in a horizontal seam in the rock, or in an open horizontal construction joint in the dam itself. Therefore, if the dam has no such planes of weakness, no consideration should be given to the possible action of uplift. On the other hand, the water pressure within the pores of the concrete exerts no influence on the static equilibrium of the structure. This view is supported by calling attention to the fact that the pressure in the pores is equal in all directions, so that to an upward pressure within a pore is opposed an equal pressure in the adjoining pores, causing all forces to be cancelled. The fallacy of these statements can be seen by referring to Professor Fillunger's scientific investigations.

If the pressure in the pores of a body is the same in all directions, but can vary from pore to pore, it is evident that no uplift can be produced. However, if the water meets greater resistance in one direction than in another an uplift pressure invariably will result. This case is present in a gravity dam, in which some parts are more porous than others. The uplift in a concrete dam depends, therefore, on the porosity of the aggregates and of the cement paste, respectively. As the difference in the porosities of these two materials in a dam is comparatively high, it follows that uplift cannot be neglected.

Since a rigid body submerged in an incompressible liquid will have an equal pressure upon its surface in all directions, the resultant of the weight of the body and of the total pressure of the liquid, acting upon it, will be directed upward or downward, depending on the relative magnitudes of these two forces. On the other hand, if the water is surrounded by a rigid body, the same resultant acts on the surface of this body but has an opposite direction. This last case is of importance in the application of this law to gravity dams. In both cases the forces acting on a surface are replaced by forces acting in space, or, in other words, by forces denoting mass.

The field intensity, p, of the pore pressure in the interior of a gravity dam is smaller than the field intensity of the water in the reservoir, that is, the hydrostatic head, which is given by,

$$p_0 = \gamma \, h \qquad (3)$$

in which,

γ = specific weight of water; and
h = height of water level measured from the point under consideration.

At the down-stream face of a porous dam the field intensity of the pore pressure will be necessarily zero, or,

$$p_2 = 0 \qquad (4)$$

Near the up-stream face the field intensity of the pore pressure will be a little less than the hydrostatic head, or,

$$p_1 < p_0 \qquad (5)$$

From this expression it follows that the water in the pores of a dam will be in a state of steady flow. Assuming that the dam is water-tight at the down-stream face as well as at the base, the infiltrating water will reach equilibrium when all pores in the concrete are filled with water. In this case the intensity of the pore pressure will be the same as the pressure in the reservoir at equal elevations. In an ordinary case, however, in which neither the dam nor its rock foundation is water-tight these lines of equal intensity or gradients will deflect toward the base and toward the down-stream face because of the variable intensities of the pore pressure (Fig. 8).

If a sufficiently small portion of the dam with the volume, *v*, is considered, then the resultant, *P*, of the water pressure on the walls of all the pores of this portion, will be expressed (according to Professor Fillunger) by,

$$P = (\mu - \nu) \frac{\partial p}{\partial z} v \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which,

$\frac{\partial p}{\partial z}$ = gradient of pore pressure,

μ = coefficient of porosity of aggregates; and

ν = coefficient of porosity of cement paste.

By dividing both sides of Equation (6) by the volume, *v*, the pore pressure per unit of volume will be obtained.

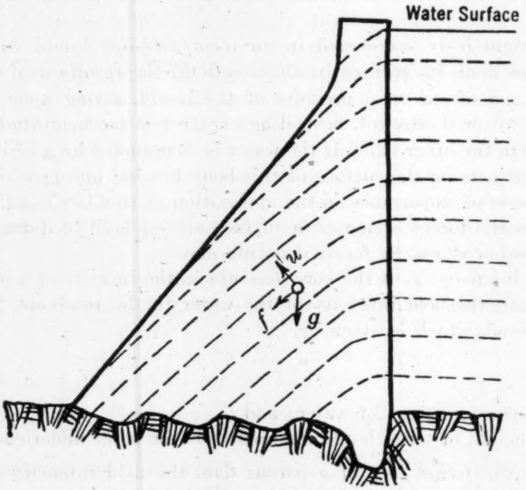


FIG. 8.—GRADIENTS OF PORE PRESSURE.

Since the porosity of the aggregates is usually less than the porosity of the cement paste, it is evident from Equation (6) that the pore pressures of this small portion will have a negative resultant. Consequently, within a dam there is always present a force with a direction opposite to that of the weight of the water. This is, however, none other than the uplift pressure itself.

Assume, for example, that it is possible to build a gravity dam with cement, but without the addition of aggregates; then, obviously, $\mu = \nu$, and Equation (6) becomes $P = 0$. In this case, therefore, uplift cannot be produced in the dam. In ordinary cases, however, the consideration of uplift in the stress analysis of gravity dams should never be omitted in spite of Mr. Wiley's assertions to the contrary. If cobblestones are to be used, this consideration is rendered more imperative, due to their larger size.

The steady flow in a gravity dam caused by the difference of pore pressure in the dam and the hydrostatic head in the reservoir will be maintained by the frictional resistance of the water on the walls of the pores. This resisting force was determined by Professor Fillunger and was found to depend on the porosity of the aggregates, on the gradients of the gravitational field (weight), and on those of the pore pressure, respectively. Therefore, within a gravity dam that is exposed to water pressure there are two forces acting, besides the weight, g. These are, the uplift, u, and the frictional resistance, f (Fig. 8). The weight is an actual force of mass with a vertical direction, the uplift is normal to the gradients of the pore pressure, and the frictional resistance is perpendicular to it. Consequently, the uplift can be considered as a normal stress, while the frictional resistance corresponds to the shearing stress. The direction of the uplift force is always opposite to that of the weight, while the frictional resistance often, but not always, has a direction the same as that of the uplift (Fig. 8).

As stated previously, the gradients of the pore pressure are inclined toward the down-stream face (see Fig. 8). Therefore, Dr. K. Kammüller has suggested omitting the horizontal component of the uplift and the entire frictional resistance.[58] By plotting the vertical components of the uplift in a horizontal section, a pressure diagram will be obtained which, in a triangular dam, is approximately a similar triangle with the zero pressure at the down-stream face. At the up-stream face this pressure is equal to the hydrostatic head.

This simplification is not quite correct, because the frictional resistance acting along the inclined gradients of the pore pressure has also a vertical component. This component, depending on whether it acts upward or downward, will make the uplift greater or smaller. However, this somewhat crude simplification of the rigorous theory indicates clearly, that the triangular distribution ordinarily assumed, of the uplift in the interior of a gravity dam, is not based entirely on arbitrary assumptions but that it has to a certain extent a scientifically justifiable meaning. It has nothing to do with the opening of a joint, as Mr. Wiley is inclined to assume, but it is the necessary consequence of the pore-pressure distribution in the structure itself.

The foregoing theory explains in a natural way the mechanical action of water in the pores of a dam and is exact and free from assumptions. It is true that the analysis depends largely on the properties of the materials used in the construction of dams and of their rock foundation for which the various coefficients of porosity, etc., must be known in advance with a reasonable degree of accuracy. However, these are problems which confront the engineer in all his activities.

---

[58] "Die Theorie der Gewichtstaumauern unter Rücksicht auf die neueren Ergebnisse der Festigkeitslehre," von Dr.-Ing. K. Kammüller, Berlin, 1929, p. 13.

In order to determine whether the water pressure exerts a bursting action within the pores of a material by overcoming its tensile strength a test specimen can be subjected to water pressure of equal intensity and, at the same time, to a pull acting along its longitudinal axis.

Rudeloff, Panzerbieter, and Professor Fillunger tested cylinders of cement mortar to failure by an axial pull.[59] In these specimens water was forced into the pores. In some specimens this penetration was prevented by means of a rubber membrane. Other specimens were subjected to a water pressure on the outer face of the cylinder. This outer surface in some specimens was also protected by a membrane; in other specimens it was not. In all cases the water pressure was raised to a certain limit and then the axial pull was increased until the material failed. In addition to these, other specimens were tested by an axial pull only without subjecting them to a water pressure.

In these tests it was found that by increasing the water pressure the decrease of the tensile strength of the material was very rapid so that under a pressure of 8 to 10 atmospheres, it was practically zero. If the results of these tests were correct, then, as Professor Fillunger states,[60] gravity dams higher than 250 ft. could not be safe, because a material with zero tensile strength will also have an uncertain compressive and shearing resistance.

The reason for this apparent difference between tests and experience is easily seen if it is realized that each of the specimens was prepared in four horizontal layers perpendicular to the axis of the cylinder. Obviously, these planes were the points of weakness. Between these layers the cement mortar was very porous so that it resisted uplift very little, while in the remaining more dense and sound parts of the cylinder the resistance was considerable. This unequal distribution of the pore pressure evidently caused the failure.

Sometimes the failure of a porous material subjected to water pressure in all directions is interpreted as being due to the bursting action of the water in the pores of the body under pressure.[61] According to Professor Fillunger, in such cases the rupture can be assigned to other causes than tension.[62] It is very probable that the release of the water pressure was too rapid so that the water in the inner pores remained under high pressure, while in the outer pores the pressure was reduced considerably. The abrupt change of the pressure within the different parts of the body causes an unequal pressure distribution which results in failure by tension.

Specimens of standard briquettes consisting of the porous Portland cement mortar, less porous slag-cement mortar, and non-porous sealing wax, were subjected by Professor Fillunger to a water pressure in all directions of 100 to 200 atmospheres, respectively.[63] Some specimens were tested without being under water pressure. Simultaneously, a steadily increasing axial pull was applied by the aid of a level arm, exactly as in testing cement mortar briquettes.

---

[59] "Versuche über den Porendruck des Wassers im Mauerwerk," von M. Rudeloff und Panzerbieter, *Mitteilungen* aus dem königlichen Materialprüfungsamt zu Gross-Lichterfelde, 1912, Ergänzungsheft No. 1.

[60] "Neuere Grundlagen für die statische Berechnung von Talsperren," Reprint, p. 10.

[61] "Permeazioni d'acqua e loro effetti nei muri di ritenuta," by Oscar Hoffman, Milan, 1928; also, "Zur Frage des Auftriebes in Talsperren," von Oscar Hoffman, *Die Wasserwirtschaft*, 1929, No. 33, and "Zur Frage des Auftriebes in Talsperren," von Paul Fillunger, ***Die Wasserwirtschaft,*** No. 4, 1930.

[62] "Auftrieb und Unterdruck in Talsperren," von Dr.-Ing. Paul Fillunger, ***Die Wasserwirtschaft,*** 1929, Nos. 18, 20, and 21, p. 334.

In all cases no appreciable changes in the tensile strength of these specimens of different porosities could be observed. The water penetrating into the pores of the body did not produce a bursting action, because otherwise under increasing water pressure the tensile strength of the material must decrease correspondingly.

Professor Fillunger's theory shows that the water pressure penetrating into the pores of the dam produces uplift and friction. The respective vectors of these forces must be combined, analytically, or by the aid of a funicular polygon, with that of the weight of the dam in order to determine the stresses produced by the water in the reservoir. It is obvious, therefore, that the water pressure in the pores of the dam is an important item which cannot be neglected in a correct analysis of stress. Consequently, Mr. Wiley's claim that stresses are not produced by the action of the water in the pores of a gravity dam—not being sustained by theory and tests—cannot be accepted as correct.

JOSEPH WRIGHT,[64] M. AM. SOC. C. E. (by letter).—During two years since 1928 dams and their safety have occupied a prominent place in engineering literature and in engineering thought. One question which has aroused some discussion and has brought out definite opinions from a few is that of initial arch resistance of arched gravity dams. Too many unknown quantities are involved in this question to warrant much confidence in a mathematical analysis; it can only be settled by observations and tests upon existing dams. Such an analysis as is possible may be somewhat enlightening, but it also serves to emphasize current ignorance on the subject in the lack of established facts to substitute assumptions founded upon more or less far-fetched inferences. Why not go at the question in a practical way? Core borings with the drill following the joints between successive blocks, and temperature readings within the hearts of well-seasoned dams, would throw much light on the subject, and other inexpensive tests could be devised that for all practical purposes would settle the question.

Little is known about the temperature changes in the heart of a thick dam; less is known about the effect and extent of water-soaking of the concrete. Engineers are equally ignorant as to the effect of the usual method of construction, that is, the casting of alternate blocks between blocks that have hardened in air, and have given up at least a small part of their chemical heat; and they know little or nothing as to "plastic flow", or "permanent set." Besides, what they know of the modulus of elasticity of concrete is worth little upon the assumption of its uniformity in a dam. Why, therefore, attempt to juggle figures that mean so little?

---

[63] "Versuche über die Zugfestigkeit bei allseitigem Wasserdruck," von Dr.-Ing. Paul Fillunger, *Oesterreichische Wochenschrift für den öffentlichen Baudienst*, 1915, No. 29; also, reprint.

[64] Chattanooga, Tenn.

The fact that the St. Francis Dam, built without vertical construction joints (as a monolith), cracked, is not conclusive to the writer. It has been well established by the "post mortem" that the abutments were incapable of sustaining heavy thrusts. This, and the fact that the dam was built as a monolith, would predicate a behavior different from that of dams constructed in the usual manner between competent abutments.

As in most questions of controversy, however, much can be said on both sides. Certainly, the St. Francis Dam should not have been built as an arched dam, with or without compensation for the convergence of radial planes, because of incompetent abutments, and its rather great length; although from all that is now known about it, it would have failed just as surely had it been so compensated. Yet no reasonable man who has seen their foundations and profiles will say that such dams as the Arrowrock, the Exchequer, and some others which have been constructed in narrow canyons, are unsafe by reason of the lack of compensation for the convergence of the radial planes; nor would he say that a straight dam of a gravity section presented less hazard as to ultimate failure. That would appear to be the crux of the question—fit the dam to the physical conditions of the site. It is well understood that a narrow canyon having competent abrupt walls is the only suitable site for a single arch of any description. Accepted principles of dam design are rooted in meager knowledge, yet it is feared that this fact is too often overlooked.

In the analysis of the gravity dam, designers know the pressure upon its face; they know, approximately, its weight; and exceedingly roughly they know the strength of concrete made under certain ideal conditions—just the bare skeleton of the problem. The remaining factors have to be assumed; yet they talk about factors of safety as if they were known definitely. In the analysis of the arched dam the engineer's confidence in reaching correct conclusions from false premises through the medium of clever mathematical gymnastics approaches the sublime! He starts with the assumption that concrete is a homogeneous material, uniformly elastic, and that it behaves uniformly under the influence of temperature changes—all of which is known to be false, or to say the least, decidedly inexact. He forms certain fundamental equations as to stresses based upon fallacies, or inexact theories of arch resistance. Then, making certain convenient assumptions with a view of simplifying the solution, he blandly proceeds to compute the stresses in all parts of the dam, ignoring the fact that an arched dam is not a true arch in any sense, and that in no two dams having different profiles would the stresses be at all comparable!

Within the last few years the tests upon models and upon the Stevenson Creek Dam, supplemented by measured strains in existing dams, have cleared the problem of the arched dam somewhat and designers now need not labor wholly in the dark; but more than anything else these tests have shown that previously accepted theories gave results widely different from the facts, and that engineers must rely largely upon models and tests of existing dams to arrive at an approximately correct design.

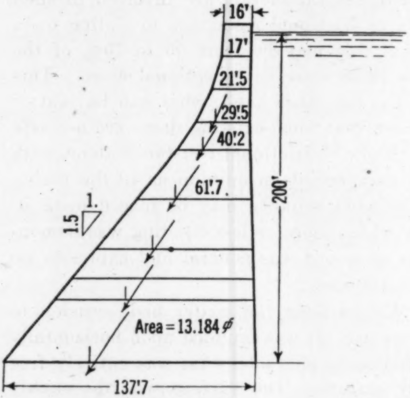
In considering the stability of dams, therefore, it is well to remember that all so-called factors of safety are based on assumptions that are generally unsupported by adequate reliable experimental data; and this would seem to be very pertinent to the question of the safety of higher and still higher dams, designed in accordance with accepted theories. The design of all dams is at present, and, for some time, must remain largely in the field of empirics, and any too radical a departure from experience is fraught with more or less danger. One safeguard that the writer would suggest is provision for drawing down the reservoir wherever at all practical, even at large expense.

Probably the majority of gravity masonry dams fail by sliding upon, or beneath, their bases. Strictly analyzed, several factors are involved in such failures of which little is known. As to frictional resistance to sliding upon the base, it is generally conceded that this may be from 55 to 70% of the pressure upon the base, or perhaps a little more in exceptional cases. This opinion seems to be fairly general. On the other hand, what can be said of bonding and of shear? It is well known that some existing dams are not safe against sliding as determined by the theory of frictional resistance alone, with 70% as the upper limit, yet few have expressed an opinion as to the probability of extensive bonding, and as to what reliance may be placed upon it. Mr. Wiley mentions two instances in which more or less bonding was demonstrated, but he fails to state whether or not it was general and expresses no opinion as to its reliability in the general case.

During the construction of the Wilson Dam the writer had occasion to build a pier which, later, had to be removed. It was founded upon horizontally bedded sound limestone of a highly siliceous character that was entirely free from cracks, and was comparatively smooth. The surface was thoroughly flaked and nicked by hammering with 4-point heavy drill-bits, and was made perfectly clean by repeated washing with high-pressure jets of air and water. A good coat of neat cement grout was then applied and broomed in and the pier was constructed of a good quality of 1:2:4 concrete. The writer was very anxious to know to what extent bonding had been effected, but was not much surprised to learn that only 4% of the area of the base had bonded with the rock surface. This too took place directly under the point from which concrete was dropped into the forms in the initial stage of construction, and there seemed no other reason why it should have bonded at this point rather than elsewhere. This point was not at the center of the pier, and the surface there was apparently no rougher than elsewhere.

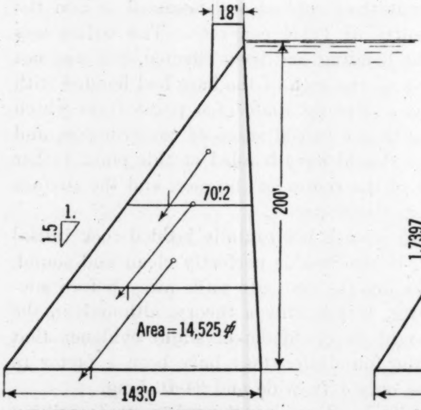
It is the writer's theory that upon smooth horizontally bedded rock initial bonding takes place quite generally, if the rock is perfectly clean and sound, but that in the process of crystallization the concrete pulls away before adequate strength is developed. However, this is only a theory, although in the case of the pier mentioned he thought he could detect slight evidence that this had happened. The shape of the foundation may have been a factor in the result. The base of the pier was only 4 ft. wide and 24 ft. long.

Advocates of the theory of extensive bonding sometimes base their opinion upon results obtained in brick and stone structures. In this, they overlook the fact that, in such a case, the mortar generally constitutes a very thin sheet and lacks the necessary cohesive strength to pull away, and that cracks form in the sheet instead. This might possibly explain the bonding observed by Mr. Wiley in the overturned sections of the St. Francis Dam. Weakness in the planes of lamination or the joint planes of the mica schist may have yielded to the cohesive strength of the concrete. However, it is easy to believe that if the foundation presents a sufficient number of prominent projections, such as would generally be the case if the stratification was upon edge, or dipped strongly, effective bonding may be obtained to a very considerable extent, with care in cleaning and grouting.


16'
17'
21'.5
29'.5
40'.2
61'.7
200'
1.
1.5
Area = 13.184 ⌀
137'.7
16'
17'
21'.5
32'
44'.3
63'.3
200'
1.
2.04
Area = 13.177 ⌀
134'.5

(*a*) ROOSEVELT DAM

(*b*) A SECTION HAVING A GREATER UPSTREAM BATTER


18'
70'.2
200'
1.
1.5
Area = 14,525 ⌀
143'.0
16'
70'.2
200'
1.
1.7397
Area = 15,085 ⌀
161'.0

(*c*) ELEPHANT-BUTTE DAM

(*d*) A SECTION HAVING A GREATER UPSTREAM BATTER

FIG. 9.—SECTIONS OF ROOSEVELT AND ELEPHANT BUTTE DAMS COMPARED WITH DAM SECTIONS HAVING GREATER UP-STREAM BATTER.

Lacking evidence as to the extent upon which effective bonding may be depended, it is necessary to rely upon frictional resistance. For this, greater pressure on the base is demanded, and the writer has often wondered why dam designers have not made greater use of up-stream batter in the case of gravity sections. Fig. 9 and Table 2 are presented to show that greater up-stream batter is not inconsistent with cantilever resistance and minimum sections. Fig. 9 (*a*) and Fig. 9 (*c*) represent sections of the Roosevelt and the Elephant Butte Dams[65] from their crests to a water depth of 200 ft. Since both these dams are much higher than 200 ft., the diagrams shown may not fairly represent stability at their bases, but since they are fairly typical of 200-ft. dam sections as usually battered, they are used.

TABLE 2.—COMPARATIVE DAM SECTIONS SHOWING THE ADVANTAGES OF UP-STREAM BATTER.

(See Fig. 9.)

Item.	Roosevelt Dam: Section above the 200-ft. water depth. (Fig. 9 (*a*)).	Section having an up-stream batter greater than Roosevelt Dam (Fig. 9 (*b*)), but having the same sectional area.	Elephant Butte Dam: Section above the 200-ft. water depth. (Fig. 9 (*c*)).	Section having an up-stream batter greater than Elephant Butte Dam (see Fig. 9 (*d*)), and an area only 4% larger.
Weight of masonry, in pounds per cubic foot	155	155	145 (assumed)	145 (assumed)
Uplift, in any horizontal plane (in percentage):				
At the heel	0	0	50	50
At the toe	0	0	0	0
Sectional area, in square feet	......	......	14 525	15 085
Factor of safety against overturning	2.260	2.263	1.93	2.32
Ratio of weight to horizontal pressure	1.680	1.839	1.378	1.626

It will be noted from Table 2 that Fig. 9 (*b*) has the same sectional area as Fig. 9 (*a*), but that it has a greater up-stream batter; yet it develops a 15.9% greater resistance to sliding and has a slightly better cantilever resistance. Fig. 9 (*d*) having a sectional area only 4% larger than that of Fig. 9 (*c*), develops a 24.8% greater resistance to sliding, and 19.5% greater cantilever resistance. The resultant of pressures on the base of Fig. 9 (*c*) falls slightly outside the middle third, under the weight of concrete assumed, whereas in the base of Fig. 9 (*d*) it falls within the middle third; hence, the difference in sectional areas.

In the face of this apparent advantage of greater up-stream batter, why is it not more generally adopted? Is the present custom a relic of the past, when such dams were generally constructed of rubble masonry, and shearing resistance was more questionable, or do designers really fear the development of cracks in the battered up-stream face? To the writer it would seem much morely likely that cracks would develop in the toe of the Roosevelt section. If cracks are not feared, then what is the real objection to such a section?

---

[65] As taken from "Construction of Masonry Dams," by Chester W. Smith, pp. 247 and 253.

Hydrostatic uplift is another question about which there seems to be no unanimity of opinion. All agree that there must be more or less uplift, but they differ as to its probable amount. Naturally this will vary greatly on different foundations, and it is not expected that this feature of design could be standardized; nor would it be desirable. Data on uplift are gradually being accumulated from measurements on existing dams, and within a few years something may be known about the subject. In the meantime, however, would it not be well to assume that with the best of foundations the uplift may be as much as full reservoir pressure at the heel and tail-water pressure at the toe, acting upon 50% of the base? In the writer's opinion this gives a dam section that is none too conservative. This, of course, is assuming that effective drainage is not provided for, or cannot be relied upon.

Mr. Wiley has touched upon leakage, or seepage along "cold joints". Leakage, or seepage, is decidedly objectionable in any dam, and is, of course, especially objectionable in thin arched dams, or others composed of comparatively thin sections. All concrete workers appreciate the difficulty of treating a cold joint so as to make it water-tight, and they know that, even with the utmost care in picking, cleaning, and grouting, some joints may be expected to seep, if not leak. One remedy, of course, is to have no cold joints. It is the writer's experience that, if the succeeding lift be cast within 24 hours of stopping, only reasonable care in cleaning, wetting, and grouting is necessary to secure water-tight joints in a moderately thick section. This, however, is not always practical, and for reasons touched upon by Mr. Henny, may not be desirable. In a recently built thin arched dam plate-steel water-stops set into the superimposed lifts were used with good results, and the writer knows no good reason why the practice should not become general—particularly in thin sections. The use of these water-stops, however, should not excuse careless cleaning and grouting.

Mr. Henny expresses a suspicion that cracks through construction blocks more or less parallel to the water face may sometimes form. The writer fears that they generally do form in dams of heavy section. The construction blocks of Wilson Dam were alternately 38 ft. and 54 ft. long on the axis of the dam, and near the base were from 75 to 90 ft. in the other dimension. The writer in many cases observed that when the upper lift had been cast about ten days or two weeks, fine cracks would appear on each side of the lift somewhere near its mid-length. These cracks could be traced part of the way across the block, from each side toward the crack in the opposite side, and the writer feels reasonably certain that some at least extended entirely through the lift. The concrete was perhaps the best the writer ever saw used in a dam. If these cracks form in blocks of this size what can be expected of blocks of a high gravity dam?

Mr. Henny suggests as a remedy that the thickness of successive blocks be limited to 4 or 4½ ft. and slower construction, with the view of holding the temperature down. It has occurred to the writer that lower lifts would be of advantage for a different reason—not to prevent cracks, but with the hope that cracks in the superimposed lifts would break joints, and would not lie in the same vertical plane. Thinner lifts would favor lower temperatures, but they would also favor more rapid cooling and drying, which would tend to produce cracks. Besides, in most cases, thin lifts would be most vexatious and costly in their effect upon speed of construction, and the greater number of "cold joints". When, in the construction of the Wilson Dam, the 12-ft. lifts were reduced to 6 ft., progress was reduced by at least 25%, and the cost of placing raised to a greater extent, although the great length of the dam was especially favorable to the use of thin lifts. In this case, since forms were handled by traveling cranes in large sections (an entire side at a time), it was easy to construct and hold the forms for a 12-ft. lift. The reduction of lifts to 6 ft., therefore, cost considerable money, and, in this case, was probably of doubtful advantage.

Shrinkage and cracking are troublesome factors to concrete designers, and it occurs to the writer that the cement manufacturer and his chemists could probably contribute much to remedy this condition. Early cracks and planes of weakness seem to form much more readily in concrete made of certain cements than in that made of other brands. Aging of the cement of course improves this condition, but it is probable that the chemist could find means more effective. The use of pre-cooled mixing water during warm weather, as suggested by Mr. Henny, would certainly contribute to the desired result.

As to temperature stresses in well-seasoned concrete, do engineers really know much about these? The coefficients of expansion of different aggregates are given by some, as follows: Limestone, 0.00000251; granite, 0.00000868; and quartz, 0.0000101. These show a wide variation. Therefore, is it not certain that concretes made of different aggregates would show almost an equal variation in coefficients of expansion, not to mention concretes of different densities made of the same aggregates?

Mr. Henny states that,

> "Concrete with a known strength of 3 000, 4 000, and even 5 000 lb. per sq. in. in 28 days is now readily secured, and there seems to be no reason why concrete in the base of a dam could not be safely subjected to compressive stresses of 1 000 lb. per sq. in., or even more (such as would obtain in a dam about 630 ft. in height), provided a source of satisfactory aggregate is at hand, and a proper mix is used."

The statement as worded is carefully guarded, but it may be well to emphasize its limitations by introducing a few more definite provisions: First, that the maximum stresses can be known within a reasonable degree of approximation and that they may not exceed this limit, temperature stresses being considered; second, that such stresses be not used in thin sections subjected to water-soaking, and to wide temperature changes; third, that designers do not deceive themselves as to the feasibility of securing the early strengths specified; and, fourth, that reliance can be placed on the permanent quality of the concrete. All these may be implied in the statement, but its limitations cannot be too strongly emphasized. However, taken as it stands, if the base of a dam of gravity section is meant, it can do no harm for the reason that, at present, economics would bar the construction of a 630-ft. dam except in narrow canyons having sound abutments, where its resistance to punching shear would be a very large factor in its stability.

As to the manufacturing of concrete and present knowledge of it, have engineers really made as much progress within the past few years as they sometimes imagine? In the writer's opinion, some of the more conservative engineers are entirely too confident of securing a specified strength. Density and water-tightness are not synonymous with strength, although they are important elements in the durability of concrete, which fact is sometimes overlooked.

Within the last several years the writer has had reason to make an occasional visit to a rather high dam that was under construction, in which high stresses were expected and for which claims of especially high-grade concrete were being made. Undoubtedly those who made such claims believed them to be true, but it was the writer's observation, confirmed by several visits, that the control exercised was entirely too lax to expect such results, and that in parts of the structure, at least, they most certainly were not secured. Some of the aggregate was too dirty for use in any structure; some of the batches that went out, appeared to be practically all mortar; occasionally, batches went out that were sloppier than the writer ever approved in the most reckless period of concrete history; and when the dam neared completion places could be pointed out where excess water was used.

In another case, the writer is told that a certain high dam which is known to have been built under rigid specifications, based upon Professor Abrams' discoveries as to strength, fineness modulus, and slump, leaks and seeps to an unusual extent.

With these experiences and others equally convincing, it is best not to be deceived in the idea that it is easy to secure a concrete of predetermined strength and density. It is not easy, and with the utmost care the results are sometimes disappointing.

Therefore, the writer advises caution in the matter of increasing the working stresses in high dams—particularly in high, thin dams. Twenty-five years hence something may be known about concrete. At present, there are very many things that engineers do not know.

A. J. Wiley, M. Am. Soc. C. E. (by letter).—Professor Switzer begins his discussion by citing three points in the main paper with which he cannot agree. The writer accepts these points as fair presentations of the claims made in his paper.

With reference to the first point Professor Switzer takes the case of a vertical slice of a dam, 1 ft. thick at the up-stream face and converging on radial planes, with full water pressure upon the up-stream face. The length of base is computed to bring the resultant, with reservoir full, at the third point for a dam, 1 ft. thick at the face, with parallel sides. He then shows that in the slice with converging radii and with the same water pressure there will be tension in the up-stream face, a conclusion for which neither argument nor calculation is needed.

Professor Switzer then calculates the deflection of the cantilever with reservoir full and concludes that it is so small that "the convergence of the radii would not close up any contraction joints such as are used ordinarily between blocks of the dam", and that, "therefore, no arch action can develop".

It would seem from the quotation that Professor Switzer believes that the contraction joints in a dam are built like those in a sidewalk, or pavement, with an actual opening filled with some plastic substance to provide for expansion. Nothing could be farther from the fact which is, of course, that concrete is tamped direct against concrete, with not even the thickness of a sheet of paper in the joint, and with the adjacent faces interlocking by offsets in one face into which the concrete of the other is poured.

Professor Switzer then states that arch action can be expected only when the contraction joints are grouted at times of low temperature, or when the arch has a comparatively short radius, and concludes that "arched action in a curved gravity dam cannot be expected."

The writer disagrees radically with both the statements and the conclusion of Professor Switzer as cited in the preceding paragraph. He is aware that there is much support for Professor Switzer's attitude in recent engineering literature, but he thinks that much of it is from prejudiced sources. In a few actual cases perhaps a very thin arched dam contracted due to the shrinkage of concrete, because it set in air, or because it was so thin that it was fully subject to seasonal or even daily temperature changes and contracted with the cold.

There may be a case in which a thicker dam has shrunk while the concrete was setting, due to a combination of very rapid pouring, an excessive use of water in the mix, or a poorly graded and unwashed aggregate.

In the vast majority of arched gravity dams there is more effective arch action than in the thinner dams which depend for stability principally upon arch action. The more effective arch action in thick arched gravity dams comes from the fact that any mass of concrete setting in air has an exterior coat which shrinks in setting because of the lack of moisture necessary for complete hydration. The interior, protected by this outer skin from evaporation, remains moist for a longer period so that hydration of the coarser part of the cement and consequent expansion of the interior mass continues long after this action is suspended on the faces.

This dried out and contracted skin, which is inert so far as arch action is concerned, is a much larger proportion of a thin arch dam than of the thick arched gravity dam so that a larger proportion of the latter is effective as an arch.

From the nature of the situation, inspection of the contraction joints in a thick dam is confined to the vicinity of the faces of the dam, and to surfaces of galleries in the dam, either of which is a case of setting in air and is apt to show cracks due to the shrinkage which follows from this condition.

There is much evidence in the many thick arched gravity dams to show that the opening of the expansion joints extends only from 10 to 20 ft. into the dam, beyond which distance the dam is practically a monolith. This is proved by the absence of leaks from the contraction joints at the downstream face; in the case of the Mulholland Dam it was shown by pressure tests.

Professor Switzer also dissents from the writer's claim that there is no uplift due to water under-pressure in the voids of the concrete. There seems to be some difference of opinion as Mr. Jakobsen and Mr. Floris have also dissented from the writer's views on this subject.

To clarify the matter it will be assumed that in some way all the voids in the concrete eventually become filled with water and that the water in the voids is under full reservoir pressure at the up-stream face, gradually reducing to tail-water pressure at the down-stream face. On a vertical column of unit cross-section at the up-stream face, assume the concrete to have a proportion of voids represented by p. In each unit of height in the column of unit area the sum of the volume of the voids will be p. It will be assumed that the voids are cubes and that they are uniformly spaced both vertically and horizontally, with n voids per lin. ft. There would be n^3 voids per cu. ft.; the volume of a single void would be $\frac{p}{n^3}$; its side or height would be $\left(\frac{p}{n^3}\right)^{\frac{1}{3}}$; its area would be $\left(\frac{p}{n^3}\right)^{\frac{2}{3}}$; and the area of all the voids in a single layer would be $n^2\left(\frac{p}{n^3}\right)^{\frac{2}{3}}$.

Assume that at any given point, H ft. below the reservoir surface, the dam is cut by an imaginary plane passing through the bottom of a layer of voids. In this set of voids the upward pressure is:

$$U = 62.5\left[\left(H - \left(\frac{p}{n^3}\right)^{\frac{1}{3}}\right] n^2 \left(\frac{p}{n^3}\right)^{\frac{2}{3}} = 62.5\,H\,n^2\left(\frac{p}{n^3}\right)^{\frac{2}{3}} - 62.5\left(\frac{p}{n^3}\right)^{\frac{1}{3}} n^2 \left(\frac{p}{n^3}\right)^{\frac{2}{3}} \ldots\ldots\ldots\ldots(7)$$

In the next set of voids $\frac{1}{n}$ above the cutting plane, the downward pressure is:

$$D = 62.5\left(H - \frac{1}{n}\right) n^2\left(\frac{p}{n^3}\right)^{\frac{2}{3}} = 62.5\,H\,n^2\left(\frac{p}{n^3}\right)^{\frac{2}{3}} - 62.5\,n\left(\frac{p}{n^3}\right)^{\frac{2}{3}} \ldots(8)$$

The uplift from water under pressure in the voids of the column of unit area at the up-stream face is the excess of the upward pressure over the downward pressure at the imaginary plane, or,

$$U - D = 62.5\,n\left(\frac{p}{n^3}\right)^{\frac{2}{3}}\left[1 - \left(\frac{p}{n^3}\right)^{\frac{1}{3}}\right] \ldots\ldots\ldots\ldots(9)$$

If $p = 0.06$ and $n = 20$, $U - D = 1\,250\ (0.0000075)^{\frac{2}{3}} \times (1 - 20)(0.0000075)^{\frac{1}{3}}$, or 0.2915 lb., which is the uplift per square foot due to water pressure in the voids. This is a constant for all heights of the dam and is seen to be negligible; but it is based upon the assumption of homogeneity in the concrete and would not apply to porous layers or any other departure from uniformity.

It appears from this that water pressure against the up-stream face does not affect the condition of stress in the concrete, but the assumption that voids are filled with water would add, for 6% of voids, 3.75 lb. per cu. ft. to the weight of the concrete. The net result of water pressure in the voids is to increase rather than to decrease the stability of a gravity type of concrete dam.

Mr. Noetzli expresses the opinion that contraction cracks parallel to the axis are likely to form in massive concrete gravity dams, and he proposes to provide for them by joints parallel to the down-stream face. The writer believes that there is no evidence of such cracks in any dam except in the open galleries where they may be purely local and due to shrinkage from setting in air. The writer recalls no indication of such cracks as those noted in Fig. 5. This diagram shows a crack in the large slab which broke from the down-stream face for some distance up from the base. The writer examined this slab very critically in the field only a few days after the failure for the purpose of finding if there was any line of demarcation between the slab and the dam from which it broke. There was positively none, and the plane of fracture was perfectly new and fresh. It was exactly such a fracture as might have occurred in a dam which was so highly stressed as a vertical cantilever as to be on the point of overturning about its down-stream toe.

There was no evidence visible to the writer of the extension of the plane of fracture above the slab to indicate it as a part of a system of cracks, as it is shown in Mr. Noetzli's section. Figs. 10 and 11 are from photographs of the St. Francis Dam taken under the direction of the State Commission of California only a few days after the failure. The photographs show very clearly all the evidence with respect to the alleged axial cracks in the dam, and close examination under high magnification convinces the writer that what Mr. Noetzli takes to be axial cracks are simply the boundaries of radial contraction cracks which had broken part way through the dam a considerable time before the failure. The writer recalls that there were areas of water-stained surface in the freshly broken concrete which were plainly due to old, nearly radial, contraction cracks which formed planes of weakness along which separation took place at the time of failure. The lines in the photograph which resemble cracks, the writer believes to be lines between the water-stained and the freshly broken concrete, and it will be observed that the most strongly marked lines are in a nearly horizontal direction ending at the up-stream face.

The writer feels that the evidence of the St. Francis Dam is certainly not convincing enough to warrant the introduction of inclined joints, with their new problems, in a concrete dam of the gravity type; nor does he think that the formation of cracks in the thin buttresses of multiple-arch and Ambursen dams, where shrinkage from setting in air occurs, is proof that similar shrinkage cracks are formed in the interior of massive gravity dams.

Mr. Noetzli's statements with respect to both the Owyhee Dam and the Boulder Canyon Dam are misleading. These structures are distinctly arched gravity dams in that they are of the same section as a straight gravity dam of equal height, but are designed so as to divide the load between the cantilever and the arch, thus reducing the stresses in both systems.

Although the writer made the statement as quoted by Mr. Godfrey that in many, if not in most, straight gravity dams, no allowance has been made for uplift, he did not mean to imply his approval of this practice. He agrees with Mr. Godfrey that uplift is a force to be recognized in the design of the gravity type, masonry dams, but he contends that where suitable physical conditions in the shape of a reasonably narrow canyon are found, security against uplift forces can be provided by designing the dam as if there were no uplift and then building it as an arch, or as an arched gravity dam. He also agrees with Mr. Godfrey in not being willing to trust the safety of a straight gravity dam against uplift to the effectiveness of cut-offs or underdrainage, although they both increase the factor of safety, at least temporarily.

However, the writer does not at all agree with Mr. Godfrey in considering uplift as the principal source of failure in gravity type dams or that the allowance for uplift in the design would remove the possibility of failure. Moreover, there has never been a failure, so far as the writer is aware, of even a straight gravity dam designed without allowance for uplift, that could not be fully accounted for by defects, either in the construction or in the foundation, so serious that even the additional weight of a dam designed for uplift, although it might have delayed, would not have prevented, its ultimate failure.

The writer does not wish to be understood as advocating the design of gravity dams without provision for uplift either by additional weight in a straight dam or preferably by the use of the arched gravity type where the topography will permit, but in his opinion Mr. Godfrey neglects the most important requirement for a gravity type masonry dam, which is that it should be founded upon a bed-rock at least as strong and durable and as truly elastic as the masonry, in the sense that it should yield only elastically to the load imposed upon it by the dam. If the foundation yields inelastically, as any material except solid rock will, and thus does not recover from the compression when the load is removed, the material does not make a safe foundation.

The writer called attention to two important elements of strength gained from building gravity dams upon a hard solid rock foundation resulting from the union which takes place between the concrete and the bed-rock. This increases the resistance to sliding on the base and also eliminates uplift on all that part of the base where the concrete is in proper contact with the rock.

Mr. Godfrey compares porous concrete to a tube made of sponge with an outer skin which is elastic and impervious and attached to the body of the sponge. He then introduces water under pressure into the interior of the sponge-filled tube and finds, of course, that the resistance of the sponge to the expansion of the tube causes tension in its fibers. The principle stated by the writer was that no uplift was caused in a homogeneous mass of concrete by water under pressure in its pores. The writer did not claim that

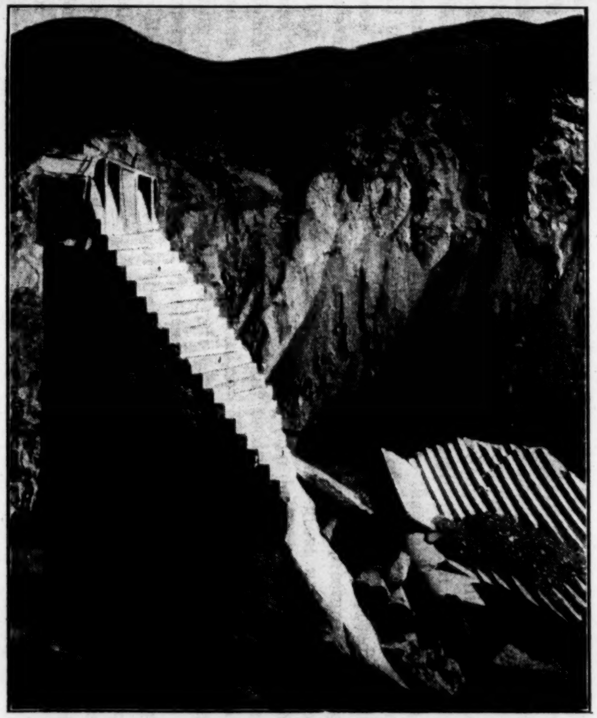


FIG. 10.—VIEW OF ST. FRANCIS DAM, SHOWING CRACKS IN DAM STRUCTURE.

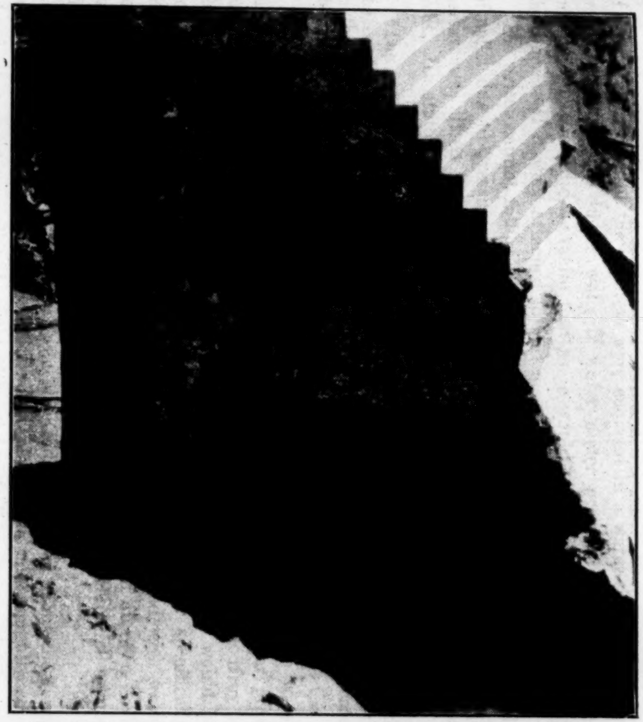


FIG. 11.—CLOSE-UP VIEW OF ST. FRANCIS DAM, SHOWING CRACKS.

if a vertical column of the concrete were enclosed in a water-tight tube there would be no tension in its walls if water under pressure were introduced into the tube; nor did he claim that there would be no horizontal tension in the concrete if it were attached to and made a part of the tube.

The writer cannot follow Mr. Godfrey's reasoning in citing as an example of uplift due to pressure in the pores of concrete, the case of a pier which he says was lifted by water pressure that entered a horizontal construction joint. He states that,

> "The great weight of the pier * * * as well as what tensile strength the concrete possessed, were not sufficient to overcome the water pressure in the pores of this concrete, which, by Mr. Wiley's statement of the case, should be balanced and equal to zero."

It is not clear whether Mr. Godfrey thinks the uplift was due to water in the horizontal construction joint as he first indicates, or to water pressure in the pores of the solid concrete as he next asserts. Mr. Godfrey is also mistaken in attributing to the writer a statement that there would be no uplift in an open horizontal construction joint. The writer specifically pointed out "that every seam or void in a masonry dam is filled with water under pressure". In another instance he stated, "in the case of an open horizontal joint in the concrete * * * there is uplift for which allowance should be made in the designs * * *".

The writer also differs radically from Mr. Godfrey's views about the function and the value of grouting. In the first place it is not generally considered as preventing uplift, but acts like the cut-off wall in reducing it. Mr. Godfrey's conception of grouting seems to be that it consists of pouring mortar into an exposed and visible opening and his statements that: (1) "It would be practically impossible to grout up a crack"; (2) "it stands to reason that solid cement or colloidal cement will not penetrate thin crevices where water may freely go"; and (3) "some of these [rock seams] may be in a horizontal direction just beneath the base of a dam, and drilling would not disclose their presence. The cracks most difficult to grout are the horizontal ones, and these are the greatest menace to the stability of the dam". All these quotations would apply if the process of grouting were to be confined to pouring mortar or grout into foundation defects exposed by the excavation, but they are not relevant to the process of "pressure grouting" which consists in forcing a thin mixture of cement grout under high pressures through drill holes extending to considerable depths below the excavated foundation. This is the process universally used for high masonry dam foundations and although the writer does not claim that it removes the possibility of upward pressure entirely, it certainly reduces it by increasing the imperviousness of the rock at the heel of the dam.

Mr. Godfrey's criticism of drainage as a relief for uplift has some justification, the writer thinks, partly because of the possibility of the sealing of the drains by mineral deposits, and partly because of the danger of deterioration of the concrete or of the foundation by the short path which it offers for infiltrating water.

In Mr. Godfrey's concluding paragraph he accuses the writer of overlooking the necessity of providing "gigantic abutments" for arched dams where such dams are not built in "rock gorges", and thinks that if the cost of such abutments were added to the cost of the dam the economy of the arch dam would vanish.

An arched concrete dam is always built in a rock gorge and every part of the dam is always supposed to be on a solid rock foundation, the sides of the gorge constituting the massive abutments which Mr. Godfrey correctly claims to be necessary.

It seems to the writer that Mr. Godfrey has failed to give proper weight to the character of the foundation and that he assumes that defects in this vital part can be remedied by increasing the dimensions of the dam to provide for uplift. The writer believes that provision should be made for uplift either by arch action or by gravity, but he does not believe that either will make a safe dam on the kind of foundation that Mr. Godfrey discusses, one in which, for instance, "excessive soil pressure due to weight * * * would merely make the dam sink", or another in which "it would be impossible for rock, no matter how poor a quality, to issue in a blow-out. Only fluid or semi-fluid soil could be forced out from beneath the base of a dam".

It seems to the writer that no better description than these quotations could be given of an absolutely unsuitable material for the foundation of a high masonry dam, and that increasing the dimensions of a masonry dam on such a foundation would only delay the time of failure.

Mr. Jakobsen objects to the writer's reference to the Exchequer Dam as an arched gravity type because in the paper an arched gravity dam has been defined as one having a section capable of carrying the full load by its weight alone and having, in addition thereto, the resistance of the massive horizontal arch. Mr. Jakobsen insists upon analyzing the gravity action of an arched gravity dam by considering the water pressure as applied to its developed up-stream face while its weight is reduced by the convergence of the radii. The writer contends that instead of applying the water pressure to the developed length of the up-stream face of the arch it should be applied to its projection upon the plane of the chord, which would give the same total water load tending to slide or overturn the arched dam as there would be on the corresponding straight dam, while the volume of the arched dam of the same section would be considerably larger and its resistance to either sliding or overturning, without considering arch action, would be greater than for the straight dam. It is when the water pressure upon an element of the curved face is considered as resisted only by the converging radial slice of that element, that there appears to be more gravity resistance in the straight dam than in an arched dam of the same section. The analysis which Mr. Jakobsen uses to show that an arched gravity dam has less stability than a straight dam of the same section, assumes the arched dam to be converted into a straight dam of length equal to the developed up-stream face of the arch, the water pressure upon the straight face being resisted by a series of independent buttresses, each consisting of a unit section touching only at the up-stream face and formed by converging radial planes. This would lead to what the writer considers the illogical conclusion that an arched gravity dam could with equal stability be expanded into a straight dam closing a valley as much wider than the actual valley as the curved face of the arch is longer than its chord. The writer's objection is that Mr. Jakobsen neglects the fact that the water pressure upon the face of an arched dam can be resolved into two components, one acting normal to the chord of the arc and resisted by sliding friction, or shear, and cantilever, or overturning moments, and the other acting parallel to the chord of the arch and causing only axial stresses in the dam.

Mr. Jakobsen contends that a transfer of some part of the load from the cantilever to the arch, even where there are no open joints extending entirely through the dam, may be prevented by a state of tension in the concrete due to shrinkage. The writer thinks it only reasonable to assume that where radial contraction joints have been provided—as is now almost universal practice—tension in the concrete between these joints may be considered as eliminated. The writer also thinks that in a relatively thick arched gravity dam shrinkage due either to temperature or to setting in air is confined to the concrete near the faces, while the interior, which is protected from temperature changes and from setting in air, tends to expand due to subsequent hydration of the coarser particles of the cement clinker, rather than to shrink in the process of setting.

Mr. Jakobsen intimates that there is an inconsistency between the writer's present position in this respect and the quotation to the effect that some years ago he considered all stresses in arched dams to be transmitted either in horizontal or vertical planes. The writer cannot see that these views are inconsistent.

The shrinkage crack in the Mulholland Dam mentioned by Mr. Jakobsen has been previously referred to by the writer as having been shown by test to be a superficial crack with a depth of less than 20 ft.

In explanation of Mr. Jakobsen's quotation from an article by the writer to the effect that if contraction joints are not provided in high concrete dams contraction cracks will form, the writer is still of the same opinion, except that later experience has convinced him that even where contraction joints have been provided in arched gravity dams the opening of the joints is superficial and probably not deeper than 20 ft.

Mr. Jakobsen questions the writer's statement that the pressure grouting of contraction joints would result only in filling them for a distance of 15 or 20 ft. from the surface, a conclusion which follows from the writer's previously discussed opinion that the opening of such joints does not extend more than this distance beyond the faces. The writer recognizes the fact that grout might be forced with an intensity sufficient to tear apart a closed contraction joint, and that in a case where, due to construction methods, open cracks are found to extend entirely through an arched dam, pressure grouting may be made to fill the entire joint. His conclusions regarding pressure grouting referred to ordinary practice.

Mr. Jakobsen asks for further explanation, illustrated by an example, of the writer's conclusion that if, as claimed by Mr. Jakobsen, the contraction joints should open so as to prevent arch action, the back pressure of the reservoir head upon the joints would compensate for the absence of arch action. The writer has already discussed the assumption previously made by Mr. Jakobsen that there might be a state of tension with no open crack, and has stated his conviction that it was untenable. In this case, too, there is the further condition that tension between open joints would not affect the result. In the case of an arched gravity dam, as the pressure in the reservoir is maintained, there must be some point where the joints are tight. The most unfavorable case for arch action would be that in which the joints were open all the way through, except for a tight seal at the down-stream face. The water pressure in the open joints normal to the faces of the radial sides of the segments between contraction joints can be resolved into components normal to and parallel to the center line of the segment. The normal components acting on the opposite sides of the segment will only cause compressive stresses in the segment. The parallel components will offset the water pressure on the projection of the converging sides of the segment, leaving as unbalanced water pressure only an area of each segment equal to its length at the down-stream face, so that each unit of the up-stream face exposed to unbalanced water pressure would be resisted by a gravity section with parallel sides, and would have the same stability as a straight gravity dam of the same section.

The writer wishes it understood that he is not advocating the construction of arched gravity dams with open radial contraction joints sealed at the down-stream face. He is simply attempting to show that, even if what he believes to be the untenable assumption—that an arched gravity dam cannot act as an arch because of open contraction joints—were correct, the dam would still have stability equal to that of a straight gravity dam of the same section because of back pressure in the open radial contraction joints.

Mr. Parsons questions whether arch action in an arched gravity dam having the same section as a straight gravity dam would offset the reduction in gravity action due to the convergence of the radii. The writer thinks he has shown in his reply to Mr. Jakobsen that, neglecting arch action, an arched gravity dam of the same section, does not have less resistance than a straight gravity dam when proper relation is maintained between the water pressure and the resistance of each dam. In fact, the arched gravity dam has the greater stability because the water pressure tending to cause sliding or overturning is the same in either case and the volume of the arched dam is considerably greater for the same vertical projection of the straight and arched dams.

The writer regrets that he has given Mr. Parsons and others who have discussed his paper the impression that he did not consider uplift with enough seriousness. This probably came from the fact that he was considering it with relation to one of his sub-headings, "Past Experience", and was trying to account for its general neglect in the past with apparent immunity from failure where other conditions were favorable. Uplift is one of the most illusive of phenomena, and it may vary under different conditions in the same dam from 100% of the head on 100% of the area to zero. A gauge tapped into a uniform concrete near the up-stream face of a dam would probably show reservoir pressure, but, as the writer has attempted to show, there would be no uplift in the body of the concrete. Gauges or pressure pipes tapped into the bed-rock at the base of the dam which show pressure head do not necessarily indicate uplift on the base, but show potential pressure which may become active pressure, or uplift, in a defective bed-rock connection, just as the potential pressure shown by the gauge embedded in the body of the concrete may become active pressure in a non-homogenous condition of the concrete, such as its intersection by an open horizontal joint or by a layer of porous concrete. For this reason uplift should be provided against, and the writer thinks that in the deep and narrow valleys or gorges where high dams are usually built, this can be done most effectively by designing the dam so as to take advantage of its resistance both as a horizontal arch and as a gravity cantilever.

The writer agrees with Mr. Werner that it would not be advisable to design an arched gravity dam with open contraction joints sealed at the down-stream face. He does not wish to be understood as advocating such a design any more than Mr. Werner advocates the design of a curved dam with a convex down-stream face just because he called attention to the fact that under certain conditions it might be more stable than a straight dam of the same section.

The writer agrees with Mr. Werner's views as to non-existence of uplift in the voids of a homogeneous mass of concrete, and as to there being an unbalanced pressure between strata of unequal porosity. The writer does not agree with the view that this would lead to uplift at a point of perfect contact between the masonry and a rock that is free from seams and voids.

Mr. Jorgensen questions the writer's conclusion that "the contraction joints of an arched gravity dam do not prevent arch action", and states that most large arched gravity dams have open contraction joints in late winter. He refers especially to the Exchequer Dam and states that it has tension in the up-stream face and that it is physically impossible for it to receive any help from arch action in the winter because of open contraction joints.

The tension in the up-stream face of the Exchequer Dam, asserted by Mr. Jorgensen, only exists upon the assumptions that there is no arch action and that its volume should be calculated with converging radii while the water pressure is considered as acting upon the developed up-stream face instead of upon its projection upon the chord. This is an assumption which has been discussed by the writer in his reply to Mr. Jakobsen.

The absence of arch action in the Exchequer Dam asserted by Mr. Jorgensen on account of alleged open contraction joints has also been discussed by the writer in his reply to Mr. Jakobsen.

Mr. Creager's discussion of the possible inadequacy of the old theories of design when applied to the much higher dams contemplated for the future, is very timely and well considered. The writer believes that the greater difficulty of providing an ample margin of safety against sliding and against tension as the height of dams is increased can be most efficiently provided by designs in which arch resistance is used in addition to gravity action.

Mr. Floris attempts to show upon the basis of the theory of Professor Fillunger, that the writer's claims with respect to the absence of uplift due to water pressure in the voids of a concrete dam are erroneous. His explanation of the theory is not entirely clear, nor is his statement of the writer's views entirely correct. His assertion that the writer limits uplift in a dam to horizontal seams in the rock foundation or to horizontal construction joints in the concrete is not consistent with the writer's claims which are that "there is no uplift in the body of a homogeneous mass of concrete, * * * nor in a perfect connection between the dam and the bed-rock, except that which may be due to a different degree of porosity in the concrete and the rock."

If the writer understands Mr. Floris correctly his interpretation of Professor Fillunger's theory is that there would be no uplift from pressure in the pores of a concrete dam unless there is greater resistance in one direction than in another, such as would result from one part being more porous than another, or from the difference in porosity of the cement paste and the aggregates. This is not essentially different from the writer's theory which is based entirely on the homogeneity of the mass of concrete. It is impossible from Mr. Floris' discussion to evaluate the amount of uplift which he claims would result from the difference in porosity of the various constituents of the concrete, but from the fact that the voids in the concrete would probably not exceed 6% of the volume and that it is only to the area of this small volume of voids that the difference in uplift (if there should be any) would apply, it seems evident that it would be a negligible amount.

The conclusion of Professor Fillunger from his experiments that dams exceeding 250 ft. in height cannot be safe, is startling, but fortunately Mr. Floris explains the apparent error by giving some details of the experiments from which it appears that cylinders made of alternate disks of tight and porous mortar were found to have practically no tensile strength in excess of that required to offset the unbalanced pressures due to water pressures of 8 to 10 atmospheres in the voids.

On the other hand, the tensile strength of homogenous samples of both porous and non-porous mortar was not affected by exposure to water pressures as great as 100 to 200 atmospheres. In the first case there was unbalanced water pressure in the pores due to alternate layers of porous and tight mortar, and in the other the pressure in the pores of the homogenous mortar was balanced so that no interior pressure affected the strength of the mortar.

So far as the writer is able to determine there is no conflict between his own conclusions and those of Professor Fillunger, except with respect to whether concrete as used in high dam construction can be considered as similar to the alternate layers of tight and porous mortar designed to give unbalanced pore pressures in Professor Fillunger's experiment first quoted, or whether it more nearly resembles the uniform mortar used in the last quoted test.

Mr. Wright gives the commonly accepted opinion of the resistance to sliding due to friction of a dam upon its base or upon seams in the rock beneath the base, as 55 to 70% of the weight. The writer believes that the most common cause of failure of straight gravity dams is sliding upon a more or less plastic seam in the rock a short distance below the base. In such cases the sliding coefficient may be very much less than the assumed minimum of 55%, but failure in that case results from a defective foundation against which no design might be able to provide.

Mr. Wright cites a case in which practically no bond was found between concrete and limestone bed-rock under a pier at the Wilson Dam. There is a theory that old concrete which has set in air forms a surface of calcium carbonate with which new concrete will not make a bond. Experience seems to confirm this theory, and Mr. Wright's discussion indicates that, in the matter of bonding with new concrete, limestone rock acts like old concrete. From the writer's experience he would conclude that with the usual care in cleaning and bonding to bed-rock, a perfect bond could be made with other than limestone bed-rock or, at least, with mica schist and sandstone.

D. C. Henny, M. Am. Soc. C. E. (by letter).—The writer has followed with intense interest the valuable discussion on high dams contributed by many experts. The attention which has thus been given to a difficult subject shows the papers grouped in the Symposium to have been opportune.

It is noted that Mr. Noetzli and Mr. Wright share the writer's opinion that there is strong probability that shrinkage cracks parallel to the water face will form in massive concrete of very high gravity dams, and that if formed they contain a serious element of danger, especially in straight gravity dams.

Mr. Noetzli defends the provision of well-drained contraction joints parallel to the dry slope contained in the specifications of the San Gabriel Dam, and although this dam may not be built, comments on this provision are none the less valuable.

Mr. Jakobsen, on the other hand, questions the wisdom of such joints in connection with a dam of unprecedented height, without, however, suggesting any substitute.

Contraction joints parallel to the down-stream slope have worked well in high buttresses of multiple-arch dams. It must be admitted, however, that these have the advantage of easy side drainage, obviating dependence on the thorough interior drainage of joints necessary in gravity dams. There is at least one precedent for such joints in an arch gravity dam, namely, the Sweetwater Dam, in Southern California, which as a result of being raised has a well-drained inclined joint in its enlarged structure. In that case the problem was complicated by the great difference in age between the old rubble masonry and the new concrete, but no difficulties appear to have resulted.

It is obvious that precautions may be advisable in dams of unprecedented height, which have not had to be used in lower structures and which are consequently an innovation. The caution advocated by Mr. Jakobsen in the use of innovations in high dam construction is very proper, provided it does not lead to the mere copying of precedent in lower structures.

Attention was called in the writer's paper to other means at hand to guard against cracking of massive concrete, namely, cooling the aggregates and mixing water or building the dam with large precast concrete blocks, to be placed in the dam after chemical heat has become fully dissipated. In summer construction, such blocks can also be pre-cooled. It is not certain that much, if any, serious extra cost is involved in the latter method, although its deviation from ordinary practice may lead contractors to bid on the safe side. At any rate, in the construction of unusually massive concrete masonry in dams, safety does demand that precautions are taken against the setting up of excessive thermal stresses in the mass and the formation of cracks which may completely change the distribution of stresses assumed in the design.

This is even more true when the tendency is considered, in the manufacture of cement, to increase the fineness of grinding, which leads to earlier development of strength and greater temperature rise. Careful study should be made of qualities of cement which will result in a reduction of temperature rise without sacrifice of ultimate strength. In that connection further research is desirable as to the effect of admixtures of siliceous substances like trass, which is understood to be freely used in Europe, and of pumicite, diatomaceous earth, and similar materials used in this country.

There is some evidence to show that heat generation would be reduced by the use of such admixtures without resulting in loss of strength. Experience with sand cement in the Arrowrock and Elephant Butte Dams, however, shows that admixtures may have an unexpected effect. In these two cases there has developed a tendency to surface disintegration. Evidently, a high degree of caution must be exercised in considering the use of admixtures on a wholesale scale in high dams.

Mr. Noetzli refers to the method of constructing the San Mateo or Crystal Springs Dam, near San Francisco, in blocks cast in place. The excellent result there obtained may be partly due to that cause, but even more important may have been the slow rate of construction progress and the relative coarse grinding of the cement, 90% passing through a 50-mesh sieve.[66]

The mere fact of casting concrete in blocks would probably not have saved the structure from cracks if it had been built with present-day speed and cement. In fact, one fine crack across the top was observed by the writer in 1906, which apparently passes indifferently through the planes separating blocks.

Mr. Wright questions the benefits which may be secured in dam construction by reducing the height of lifts as against the admitted disadvantage of more numerous construction joints. He admits that thinner lifts would favor lower temperature, but fears as a result of rapid cooling and drying, that through shrinkage cracks would form. There need be no question of rapid drying as it is becoming common practice to require concrete surfaces to be maintained moist for an extensive period, and especially the surfaces of lifts until covered by the succeeding lift. The average time of exposure is usually not very long and even where there has been seasonal suspension of work, the writer has never observed cracks in the top surface of lifts where they were wet.

---

[66] *Engineering News-Record,* May 8, 1930, p. 768.

On the other hand he does not share the hope expressed by Mr. Wright that thin lifts would favor a break of joints in crack formation in successive lifts—the adherence between lifts and the tendency in concrete for cracks once formed to extend into the adjoining mass would prevent cracking in steps.

Vertical cracks approximately parallel to the water face and near the toe may introduce immediate danger in straight gravity dams. In arched dams they are strenuously to be avoided, but they do not introduce the same element of danger because they need not interfere with arch action.

Mr. Noetzli refers to the Owyhee and Boulder Canyon Dams as being of the single arch type. He is correct if it is understood that they are both designed on the assumption that loads are carried partly by arch and partly by cantilever. To insure such division of load it is necessary that especial attention be paid to available methods of minimizing thermal stresses.

Mr. Godfrey states that it is standard practice to neglect under-pressure in gravity dam design. This may have been true in the past, but since 1920, or even as far back as 1910, it is believed that this standard has changed. He quotes the writer as having given the impression that under-pressure is easily averted or avoided. This is hardly in harmony with the fact that uplift was given considerable prominence in the writer's paper, nor with the statement occurring therein that, "generous allowance for uplift in the design * * * are the only safeguards, * * *."

It may also be stated that the writer has deemed the subject of such far-reaching importance that he has urged experimental determination and research wherever practicable. The knowledge thus far gathered shows in two ways that assumptions concerning uplift in recent dam design have been ample. Uplift pressures as measured have been less in each case, where tested, than allowances made in the design; and investigation indicates that the assumption that such pressure is effective over 100% of the base, is very conservative. There is good reason to believe that with careful construction this percentage is less than 40, although this opinion should not lead to cutting down the present common assumption of 100% effective uplift area because it is in this matter of uplift that an ample factor of safety is most necessary.

Mr. Godfrey fears blocking of drains by freezing at the outlet and serious ineffectiveness of drains except in their immediate vicinity. The writer has never observed such effect of freezing and believes that, where drains discharge into open gutters in drainage galleries (as is now usually the case), no danger from freezing can exist.

Research as to uplift also indicates uniformly the reasonable effectiveness of properly spaced drains wherever points to measure uplift pressures have been installed. It is, however, of value to have present-day practice questioned as to effectiveness, because this should lead to closer attention to details. One requirement should be that drains must be straight and easily accessible for cleaning, and this detail is too often disregarded.

An interesting question raised by Mr. Godfrey is that of buoyancy in the case of concrete resting on a foundation below the level of back-water. It has been the writer's practice to consider full buoyancy. He believes, however, that no complete buoyancy due to tail-water can exist, because of the adherence of concrete to rock, or to concrete, and that its assumption in itself contains a considerable element of safety.

Mr. Godfrey's statement that tests for high dams have shown under-pressure of practically 100% back-water pressure after sufficient time has elapsed, is admitted where any pressure is found to exist. It is the area over which this pressure acts that remains uncertain.

The term "standard dam design" has been applied by Mr. Godfrey to arching gravity dams without consideration of the ability of the abutments to take arch thrust. The writer cannot agree to this and knows only of a few cases where this charge might hold true, the most notorious of which is the St. Francis Dam.

Mr. Jakobsen argues that "gravity dams should be designed as such, irrespective of arch action, and that arch dams should be designed as such irrespective of gravity action." Such conclusion appears to the writer to be ultra-conservative. There seems to be no reason why an analysis of stresses, such, for instance, as that by the trial-load method, taking into consideration temperature drop, tangential shear, and twist, should not be made the basis of design.

The complexity of the calculations might tend to create doubt as to the dependence to be placed on the result. Yet the highly satisfactory coincidence between calculated and estimated deflections in models establishes confidence in the reliability of the method. The uncertainty as to the elastic qualities of the concrete used and the seemingly unavoidable errors in some of the underlying assumptions make it necessary to limit stresses to a fraction of known strength.

Mr. Jakobsen's statement concerning the Crystal Springs Dam is welcomed. The dam has not been brought up to its maximum intended height and, therefore, has great excess strength. This statement should have been included in the original paper, but was unintentionally omitted.

As to cracks parallel to the water face in gravity dams, Mr. Jakobsen suggests that some cracks noted in galleries may extend only a short distance into the mass. This fact is difficult to determine, although where water oozes from such cracks the probability may well be that it extends for a considerable distance. One crack was observed a few years ago in the Elephant Butte Dam, in one of the cross galleries. Its position in this case robs this fact of serious significance, as it occurs about 10 ft. down stream from the main gallery and, consequently, far up stream from the toe, where such cracks would be most dangerous. It is significant that water slowly oozing from this crack indicates that the vertical drainage system is not 100% effective. The extent or depth of this crack is unknown.

In the Bull Run Dam a crack parallel to the water face has been observed in one of the cross galleries only 40 ft. from the toe. It is very fine and extends across the gallery, but on neither side of the gallery does it reach the floor level; nor is there any visible crack in the floor. Careful scrutiny also failed to reveal any crack in the surface of the dam slope above its appearance in the gallery, and the strong probability is that the gallery crack is superficial in character and due to rapid air cooling and drying.

The fact that uplift under the base of dams may exist on planes having a down-stream pitch, and that its horizontal component constitutes an addition to the horizontal water loading, is a valuable contribution to the discussion from Mr. Parsons. The argument applies equally to uplift which may exist along construction joints within the body of a dam and indicates the desirability of giving construction joints in concrete an upward pitch in a down-stream direction.

This expedient as regards the down-stream half of the dam was adopted in the construction of the Bull Run Dam and is also applied to the gravity abutments of the Ariel Dam, on the Lewis River, in Washington. In both cases the more important object in mind was to avoid flat construction joints along which sliding might take place.

Mr. Jakobsen very properly calls atttention to the error which may be involved in the assumption that the vertical stress distribution across a horizontal plane in a concrete dam is linear. The 1908, the Ottley and Brightmore, as well as the Wilson and Gore, experiments with models, previously mentioned, indicate that this assumption, while it may be approximately correct at higher elevations, is in error near and at the base. In themselves, these experiments do not show that curved distribution of vertical stresses causes the maximum stress in a horizontal plane to exceed the maximum found with linear distribution. Nor do they necessarily indicate tension at the water face where compression was previously found; but they raise a serious doubt. This doubt is understood to be confirmed as a result of Mr. Jakobsen's application of the theory of least work to this problem.

These uncertainties may well have a bearing on the justifiability of arching a gravity dam in regard to which Mr. Werner and Mr. Hanna argue that additional expense is not warranted. To the writer it seems that, whether or not a curved gravity dam, after cooling, acts in all its parts as an arch dam when loaded, is not the only question. A relatively high dam will probably do so, in various parts of its height, as a result of cantilever deflection. The behavior will depend on numerous factors, especially temperature drop after arch closure or after pressure grouting of contraction joints. However, assuming that the dam does not initially act as an arch and that actual down-stream movement on its base or on higher planes must take place to put arch action into full play, and considering that it is only by sliding that gravity dams have failed, then it appears that, if the abutments are capable of taking arch thrust, a greater degree of safety must result from arching a gravity dam to a reasonably effective radius. This added margin of safety is, in the writer's opinion, worth considerable extra expense, at least until the forces due to uplift and those opposing sliding, as well as the interior stresses in the concrete, especially near the water face, can be estimated with greater certainty than at present.

The writer agrees with Mr. Jorgensen that there is no reason why an arch dam should not be built as high as any gravity dam on sites where it would fit—that is, where the nature of the rock abutments as well as the topographic conditions are suitable.

Mr. Creager argues that the tensile strength of concrete at the up-stream face of a gravity dam, as well as the shearing strength along sliding planes, if treated as margins of safety, are of far smaller relative value in a high than in a low dam. This argument is sound and should lead to the adoption of relatively safer sections for dams of increased height.

Mr. Hazen performed a real service in urging increased caution in the design of dams of unprecedented height.

J. G. Tripp, M. Am. Soc. C. E. (by letter).—The construction of dams involves both skill and judgment to create in finality the thought of the designing engineer. Drawings and specifications when well done, attempt to delineate and determine, accurately, the masses to be opposed to the water.

The writer has confined his efforts to the technique of construction, and the technology of temporary structures for use in carrying out the designer's intention for the permanent dam. In constructing several dams he has observed a wide divergence of opinion between designers as to the effect of concrete cracking and uplift pressures. Each designer will set forth his best conclusion in the plans and specifications, which to the builder are often enlightening and sometimes contentious and disappointing. It is unfortunately true, in the writer's opinion, that many dams are cheapened for financial reasons beyond the limits of sound engineering. He has observed all too many such structures; however, he is also sensible to the fact that an engineer worthy of the name must "do with one dollar what any one can do with two."

Because of capital investment the tendency to-day in concrete dams is to design for strength at the sacrifice of impermeability, particularly in high dams where the cubic yardage is a progressive ratio function of the height. This is a mistake. The addition of pumice to produce lower permeability and overcome laitance deposits should be encouraged. It is claimed that the addition of pumice will reduce shrinkage which, of course, is much to be desired and investigations to prove this point should not be delayed.

Shrinkage cracking in the mass can be controlled to a great extent by attention to water-cement ratio, proper relationships in aggregates and their size, and by the methods and times of deposit in the forms. The addition of ample reinforcing steel, while costly, is effective. Properly embedded steel will last indefinitely. The shape of the pouring block and its dimensions and speed of construction are matters of controversy, and, even with such a splendid example as the Crystal Springs Dam, some substantial experimental work should be done in this field to determine the proper geometrical shapes before any decision as to good practice should be adopted.

The writer agrees with Mr. Noetzli that the theory of principal inclined stresses is valid. During the construction of several important dams, he has seen the actuality of dangerous cracking, which could have been controlled by the application of this theory in the design. The writer has seen cracks develop rapidly both transversely and longitudinally during construction of large and small dams, in speedily as well as in slowly built work. He concludes, therefore, that speedy construction is not necessarily a concomitant cause for shrinkage cracking. Different cements and aggregates and water have marked effects, as do times of day in which the concrete is poured. Protection from sun and wind and sufficient curing water applied between initial and final setting minimizes "drying out" or primary shrinkage.

The writer's observations, working under the specifications of eminent authorities have led him to conclude that while many of the vagaries of construction may be blamed for partial failures, there is still a large amount of guessing as to what constitutes good practice, and not enough conclusive experimental data available as to what is good practice in the matter of concrete for either gravity or other types of dams. Among other things, he believes that to specify never less than a barrel of cement to the yard is good practice.

In the matter of foundation uplift, the writer believes that the structural geology of the sites answers grouting criticisms. Basalt will require a different treatment from an upturned sedimentary or a massive felsite. The writer has seen large masses of sedimentary rock disturbed by even low-pressure grouting, as suggested by Mr. Parsons. The writer agrees with other observers that uplift on foundations is a very potent destructive force and in all cases, under his observation, it has been considered in, or nullified by, the design. He does not agree, however, that proper drainage and either cement or asphalt grouting cannot control this force to a large extent.

Uplift between construction joints is easily controlled by horizontal copper water-stops, proper joint cleaning, and pouring of succeeding layers of concrete. It is certain that the contact between a thoroughly cleaned bed-rock and concrete is always tight and that uplift is of no possible consequence in that plane. The joints and seams below the dam are the danger spots. The shaping of the foundation assuredly has the effect desired by the designer, where the rock masses are suitable. Resultant forces can be directed to advantage by careful attention to this detail.

Finally, the construction of the dam must be manipulated and controlled with adequate and necessary plant. The fitting of plant to design conditions and specification limitations, should be understood by the directing engineer to the end that design and construction should work to the final result of safety and economy.

Without a better understanding of the hidden facts to be revealed by further investigation, it does not seem wise to predict either the type or the height of future dams.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1767

ADJUSTMENT OF TRANSIT AND STADIA TRAVERSES[1]

By HOWARD S. RAPPLEYE,[2] ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. LOWELL O. STEWART, PAUL H. UNDERWOOD, F. H. PETERS, H. PARRY, AND HOWARD S. RAPPLEYE

SYNOPSIS

This paper presents the problem of adjusting transit and stadia traverses by correcting the lengths only and sets forth an extremely simple method of applying the theory of least squares to the problem. The method suggested is so simple that it should be readily understood and utilized by any one at all familiar with plane surveys and their computation.

THE PROBLEM

The transit and stadia method of traversing has come to be used very largely by the Civil Engineering Profession for preliminary, topographic, and for many other kinds of surveys. When traverses, run by the transit and stadia method, are closed upon the initial station, or when they are run between widely separated control points established by surveys of a higher order of accuracy, some form of adjustment is necessary to eliminate from the computed co-ordinates of the instrument stations the effect of errors of closure.

In both compass-and-chain and transit-and-tape surveys the angle and distance measures are similar enough in degree of accuracy to permit the use of a method of adjustment which places corrections on both angles and lengths. In traversing by the method of the transit and stadia, however, the angle measures far surpass the distance measures in the matter of accuracy. For this reason it seems desirable to adjust this type of traverse by a method which will place corrections on the lengths only.

With the modern transit, reading to single minutes, or even less, the angle measurements around a traverse can easily be made with such accuracy that the angle closure will be very small. The angular error can be eliminated by the occasional application of a correction of one in the last place. The discrepancies most likely will have occurred in angles measured between the shorter sides of the traverse, and corrections should preferably be applied to such angles. This method leaves the error of closure, or "failure", in latitude and departure, to be adjusted by placing corrections on the lengths alone and will not change the angles or bearings measured or arrived at by this arbitrary process of distribution.

---

[1] Published in November, 1929, *Proceedings*.

[2] Assoc. Mathematician, Div. of Geodesy, U. S. Coast and Geodetic Survey, Washington, D. C.

The purpose of this paper is to set forth such a method of adjustment and to carry through a sample adjustment by way of illustration. The method is rigid, being a least squares adjustment,[3] but is presented in a more or less "rule-of-thumb" manner.

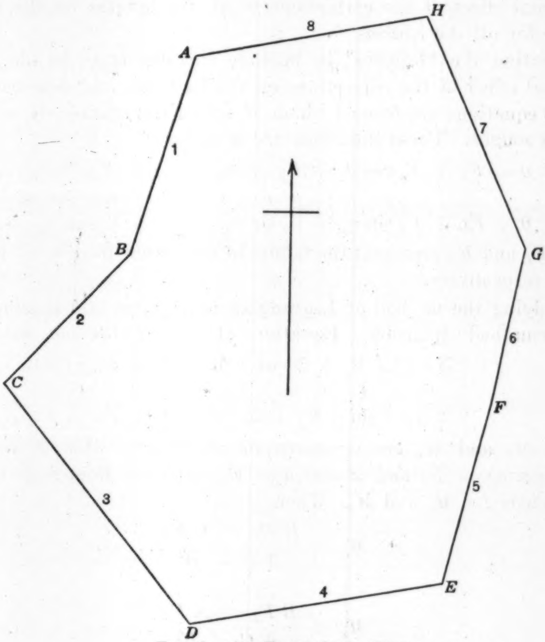


FIG. 1.—AN EIGHT-SIDED FIELD.

METHOD OF ADJUSTMENT

The traverse which it is proposed to adjust is shown in Fig. 1. In Columns (1), (2), and (3) of Table 1 will be found the observed data for the traverse. Columns (4) and (5) contain the cosines and sines, respectively, for the bearings given in Column (2). The latitudes and departures given in Columns (6) and (7) are computed in the usual manner, and the column totals represent the failure of the survey to close in latitude and departure.

---

[3] The complete mathematical proof supporting this paper has been filed for reference in the Engineering Societies Library, 33 West 39th St., New York, N. Y.

Since the traverse is to be adjusted by placing the corrections on the lengths, the correction to Course 1 may be represented by the symbol, V_1, the correction to Course 2 by the symbol, V_2, and so on throughout the whole traverse.

If the bearings of the courses are represented by the symbols, a_1, a_2, a_3, etc., then, $V_1 \cos a_1$ will represent the effect of the correction to Course 1 on the latitude of Course 1. Similarly, the expression, $V_1 \sin a_1$, will represent the effect on the departure of Course 1 of the correction to Course 1. Similar expressions naturally follow for each of the other courses.

Then, $V_1 \cos a_1 + V_2 \cos a_2 + \ldots + V_n \cos a_n$ will represent the total effect of the corrections to all the lengths on the sum of the latitudes for all the courses. Similarly, $V_1 \sin a_1 + V_2 \sin a_2 + \ldots + V_n \sin a_n$ will represent the total effect of the corrections to all the lengths on the sum of the departures for all the courses.

By equating the "failures" in latitude and departure to the expressions for the total effect of the corrections on the latitudes and departures, respectively, two equations are formed which, if solved simultaneously, will give the corrections sought. These equations are as follows:

$$0 = F_L + V_1 \cos a_1 + V_2 \cos a_2 + \ldots + V_n \cos a_n \ldots\ldots\ldots (1)$$

and,

$$0 = F_D + V_1 \sin a_1 + V_2 \sin a_2 + \ldots + V_n \sin a_n \ldots\ldots\ldots (2)$$

in which, F_L and F_D represent the failure of the survey to close in latitude and departure, respectively.

By applying the method of Legrangian multipliers and making all necessary mathematical expansions,[3] Equations (1) and (2) become, respectively:

$$\Sigma \cos^2 a \, M_c + \Sigma \cos a \sin a \, M_s + F_L = 0 \ldots\ldots\ldots (3)$$

and,

$$\Sigma \cos a \sin a \, M_c + \Sigma \sin^2 a \, M_s + F_D = 0 \ldots\ldots\ldots (4)$$

in which, M_c and M_s are constants or multipliers. Let $\Sigma \cos^2 a = A$; $\Sigma (\cos a \sin a) = B$; and $\Sigma \sin^2 a = C$, and solve Equations (3) and (4) simultaneously for M_c and M_s. Then,

$$M_c = \frac{B F_D - C F_L}{A C - B^2} \ldots\ldots\ldots (5)$$

and,

$$M_s = \frac{B F_L - A F_D}{A C - B^2} \ldots\ldots\ldots (6)$$

While the solution of this problem is a matter involving the Theory of Least Squares, in this particular case it is possible to make the solution of the two equations and the computation of the needed corrections to the observed lengths in a purely mechanical manner, as illustrated in Table 1.

The numerical values of the multipliers, M_c and M_s, are found by substituting the values of F_L, F_D, A, B, and C, which are given as totals in Columns (6), (7), (8), (9), and (10), respectively. The desired corrections to the observed lengths are then represented by the expressions:

$$\left.\begin{aligned} V_1 &= M_c \cos a_1 + M_s \sin a_1 \\ V_2 &= M_c \cos a_2 + M_s \sin a_2 \\ V_n &= M_c \cos a_n + M_s \sin a_n \end{aligned}\right\} \ldots\ldots\ldots (7)$$

TABLE 1.—COMPUTATIONS FOR CLOSING AN EIGHT-SIDED FIELD.

COURSE.						LATITUDE.	DEPARTURE.									ADJUSTED.	
No.	Station to station.	Bearing (a).	Distance, in feet.	cos a.	sin a.	N. (+). S. (−).	E. (+). W. (−).	$\cos^2 a$.	$\cos a \times \sin a$.	$\sin^2 a$.	$M_c \cos a$.	$M_s \sin a$.	$M_c \cos a + M_s \sin a$.	Correction to observed distance, in feet.	Adjusted length, in feet.	Latitudes.	Departures.
(1)	1(a)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
1	*A-B*	S. 18° 26′ W.	802	−0.94869	−0.31620	−760.8	− 253.6	+0.900	+0.300	+0.100	+1.031	−0.520	+0.511	+0.5	802.5	−761.3	− 253.7
2	*B-C*	S. 45° 00′ W.	720	−0.70711	−0.70711	−509.1	− 509.1	+0.500	+0.500	+0.500	+0.769	−1.162	−0.393	−0.4	719.6	−508.8	− 508.8
3	*C-D*	S. 39° 17′ E.	1 202	−0.77402	+0.63316	−930.4	+ 761.1	+0.599	−0.490	+0.401	+0.841	+1.040	+1.881	+1.9	1 203.9	−931.8	+ 762.3
4	*D-E*	N. 80° 32′ E.	1 029	+0.16447	+0.98638	+169.2	+1 015.0	+0.027	+0.162	+0.973	−0.179	+1.621	+1.442	+1.4	1 030.4	+169.5	+1 016.4
5	*E-F*	N. 15° 57′ E.	615	+0.96150	+0.27480	+591.3	+ 169.0	+0.924	+0.264	+0.076	−1.045	+0.452	−0.593	−0.6	614.4	+590.7	+ 168.8
6	*F-G*	N. 14° 02′ E.	701	+0.97015	+0.24249	+680.1	+ 170.0	+0.941	+0.235	+0.059	−1.055	+0.398	−0.657	−0.7	700.3	+679.4	+ 169.8
7	*G-H*	N. 24° 27′ W.	1 025	+0.91032	−0.41390	+933.1	− 424.2	+0.829	−0.377	+0.171	−0.989	−0.680	−1.669	−1.7	1 023.3	+931.5	− 423.5
8	*H-A*	S. 79° 42′ W.	948	−0.17880	−0.98389	−169.5	− 932.7	+0.032	+0.176	+0.968	+0.194	−1.617	−1.423	−1.4	946.6	−169.2	− 931.3
Totals.	......	....	......	....	....	+ 3.9	− 4.5	+4.752	+0.770	+3.248	....	....	....	....	....	0.0	0.0

and their calculation is tabulated in Columns (11), (12), and (13) of Table 1. The adopted corrections, to the nearest tenth of a foot, are entered in Column (14). These are applied to the observed lengths to obtain the adjusted lengths in Column (15) and the adjusted latitudes and departures are computed as before, except that the adjusted lengths are used instead of the observed lengths.

If desired, the adjusted latitudes and departures may be computed by multiplying the corrections to the lengths in Column (14) by the cosines and sines, respectively. This will produce corrections which, when applied to the preliminary latitudes and departures, will give the adjusted values.

Special care must be taken throughout the entire calculation to see that the proper algebraic sign is used with each tabular entry, and in computing the values for M_c and M_s a rigid observance of all algebraic signs is necessary.

In cases in which it is desired to fit a traverse to two separated control points the fixed course of the line between the control points should be put into Table 1 as the closing side of the traverse. This course, then, will not be assigned a correction, and Columns (8) to (15), inclusive, will contain no entries on the line for the fixed course. The latitude and departure of the fixed course will be the same in the preliminary and adjusted tabulations.

Most of the calculation required in this method of adjustment (Columns (8) to (14), inclusive), can be performed with a slide-rule and the whole process is so extremely simple that it involves no mathematical "gymnastics" that are beyond the grasp of even the beginner in the art of surveying.

DISCUSSION

LOWELL O. STEWART,[4] Assoc. M. Am. Soc. C. E. (by letter).—The writer agrees with Mr. Rappleye that it is desirable to place the corrections for transit and stadia traverses in the lengths only; but he would add a few words in the way of caution lest one overlook some of the field conditions that may have an important effect on the results.

In the first place the example chosen seems to be at the very upper limit of accuracy in error of closure. Probably such accuracy would not be obtainable by ordinary methods with many of the transits in common use. As a rule the probable variation either way from the chosen reading is much more than 1 ft. at a distance of 1 000 ft.

Moreover, this variation is a sort of systematic error which seems to depend upon the individual. It has been found in many kinds of instrument work that an observer tends to "under-read" or to "over-read", on each observation. He rarely alternates. This seems to be true of stadia readings. The discrepancy will vary, approximately, with the distance because, as the length of sight increases, the intervals between the larger divisions would bear the same apparent relationships to one another as did the more minute divisions at the shorter distances. It would be wise, then, for an engineer who proposes to make an important stadia traverse to test his readings at various measured distances.

It is assumed, of course, that the stadia interval is known accurately. Many transits are functioning with home-made cross-hairs and the interval may or may not be the conventional "100", which is commonly used.

Atmospheric conditions have an important effect on stadia readings. On long sights the line of sight must pass near the ground where the refraction is uncertain and the readings are correspondingly unreliable. In the same way a line over a plowed field on a hot day would be differently affected from a line over a pasture, or over a body of water.

All the foregoing factors have an important bearing on any adjustment. In fact, their effect should be reckoned and applied before an adjustment is attempted. Unhappily, some of these factors are not always known with accuracy. In the latter cases one should be cautious about placing too much faith in a rigorous method of adjustment, such as the author has set forth in a thorough manner.

PAUL H. UNDERWOOD,[5] M. Am. Soc. C. E. (by letter).—The author has well assumed as a basis for the application of the method of least squares to this problem that the corrections applied to the latitudes and departures should not affect the azimuths or bearings of the lines of the traverse, and the method devised seems to be as simple as any that can be used for this purpose.

It has occurred to the writer, however, that an even better distribution of error, from the theoretical standpoint, is obtained by considering the corrections to vary as the lengths of the lines of the traverse. This may be done by introducing weights for the various courses. If these weights are considered to vary inversely as the lengths, the solution can be carried out with no more labor than by Mr. Rappleye's method. The necessary changes in his equations are indicated as follows.

---

[4] Asst. Prof., Civ. Eng., Iowa State Coll., Ames, Iowa.

[5] Prof. of Surveying, School of Civil Eng., Cornell Univ., Ithaca, N. Y.

The latitude of a course is designated by L, and the departure by D. Furthermore, let $\Sigma\ (L \cos a) = A'$; $\Sigma\ (D \cos a) = B'$; and $\Sigma\ (D \sin a) = C'$.

With the weights varying inversely as the lengths of the courses the following equations may be derived to replace Equations (3) and (4), respectively:

$$A'\,M_c' + B'\,M_s' + F_L = 0 \qquad (8)$$

and,

$$B'\,M_c' + C'\,M_s' + F_D = 0 \qquad (9)$$

The solution of these equations for M_c' and M_s' gives:

$$M_c' = \frac{B'\,F_D - C'\,F_L}{A'\,C' - (B')^2} \qquad (10)$$

and,

$$M_s' = \frac{B'\,F_L - A'\,F_D}{A'\,C' - (B')^2} \qquad (11)$$

The desired corrections to the observed lengths are, therefore:

$$\left.\begin{aligned} V_1 &= M_c'\,L_1 + M_s'\,D_1 \\ V_2 &= M_c'\,L_2 + M_s'\,D_2 \\ V_n &= M_c'\,L_n + M_s'\,D_n \end{aligned}\right\} \qquad (12)$$

TABLE 2.—COMPUTATIONS FOR CLOSING THE EIGHT-SIDED FIELD SHOWN IN FIG. 1.*

COURSE.		$L \cos a$.	$D \cos a$.	$D \sin a$.	$M_c'\,L$.	$M_s'\,D$.	$M_c'\,L + M_s'\,D$.	Correction to observed distance, in feet.	Adjusted length, in feet.	ADJUSTED.	
No.	Station to station.									Latitudes.	Departures.
(1)	1(*a*)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
1	*A-B*	+ 722	+240	+ 80	+0.84	−0.39	+0.45	+0.4	802.4	−761.2	− 253.7
2	*B-C*	+ 360	+360	+ 360	+0.56	−0.79	−0.23	−0.2	719.8	−509.0	− 509.0
3	*C-D*	+ 722	−587	+ 482	+1.03	+1.17	+2.20	+2.2	1 204.2	−932.0	+ 762.4
4	*D-E*	+ 28	+167	+1002	−0.19	+1.57	+1.38	+1.4	1 030.4	+169.5	+1016.4
5	*E-F*	+ 570	+162	+ 46	−0.65	+0.26	−0.39	−0.4	614.6	+590.8	+ 168.9
6	*F-G*	+ 662	+165	+ 41	−0.75	+0.26	−0.49	−0.5	700.5	+679.6	+ 169.9
7	*G-H*	+ 847	−386	+ 196	−1.03	−0.66	−1.69	−1.7	1 023.3	+931.5	− 423.5
8	*H-A*	+ 30	+167	+ 917	+0.19	−1.54	−1.35	−1.4	946.6	−169.2	− 931.4
Totals.	....	+3941	+288	+3124	....	....	....	....	....	0.0	0.0

---

* As stated in the text, Columns (2) to (7), inclusive, of the author's Table 1, have been omitted in Table 2.

To illustrate this method, Table 2 shows the computations of the same traverse for which the calculations by the author's method were shown in Table 1. In the first eight columns, that is, to and including Column (7), the tabulations are the same for the two methods, and are not repeated in Table 2. The quantities in Column (8), (9), and (10), Table 2, were computed by the use of the ordinary 10-in. slide-rule. It may be noted that all the signs of all the quantities in Columns (8) and (10) must necessarily be positive. The footings of the three columns are, respectively, A', B', and C'. Substitution in Equations (10) and (11) gives -0.001104 and $+0.001543$, respectively, as values of M_c' and M_s'. By use of the slide-rule these values are multiplied by the latitudes and the departures, respectively, the calculated values being entered in Columns (11) and (12). The computations from that point are similar to those in Table 1.

A comparison of results by the two methods shows in this case no very great differences. One reason is that the courses do not vary markedly in length. A close examination reveals, as might be expected, that the corrections to the longer lines are in general larger and those to the lengths of the shorter lines are smaller by the writer's method than by that of the author. To get a comparison which would indicate more clearly the variation in the results by the two methods, the writer has taken another traverse in which there is a greater variation in the lengths of the courses. The resulting corrections are shown in Table 3.

TABLE 3.—COMPARISON OF CLOSURE CORRECTIONS FOR FIELD CONTAINING SHORT AND LONG SIDES.

COURSE.		Bearing.	Distance, in feet.	Correction to observed distance, in feet, by author's method.	Correction to observed distance, in feet, by writer's method.
No.	Station to station.				
1	*K-L*	N. 23° 55′ E.	528	—0.3	—0.2
2	*L-M*	N. 38° 36′ W.	540	—2.1	—1.7
3	*M-N*	N. 50° 43′ E.	671	+0.7	+0.7
4	*N-O*	N. 78° 51′ E.	375	+1.6	+0.9
5	*O-P*	S. 20° 25′ E.	868	+1.8	+2.3
6	*P-Q*	S. 19° 38′ W.	447	+0.5	+0.3
7	*Q-K*	S. 79° 53′ W.	937	—1.6	—2.3

It will be noted that the variations in the corrections are more apparent, the writer's method, based on weights, giving for the shorter courses smaller corrections and for the longer courses larger corrections. This is particularly noticeable when Courses Nos. 4 and 7 are compared, the method without weights leading to equal corrections for these two courses, while the method with weights gives a correction of +0.9 ft. for the shorter course and —2.3 ft. for the longer one. Since these two courses are nearly parallel, the variation is not due to other factors to any extent, but is due almost entirely to the weighting.

In conclusion, it may be said that, where all the courses are very nearly of the same length, there will be little variation in results by the two methods. Where there is considerable variation, the results will be noticeably different. Since the calculations are no more lengthy or difficult by the method with weights, and since this gives results which, on theoretical grounds, are superior, it would seem that this method is preferable.

F. H. PETERS,[6] M. AM. SOC. C. E. (by letter).—This paper deals with a problem that has been the subject of special investigation by the Topographical Survey of Canada, which has probably done more of this class of work than any other survey organization.

The rapid development of Canada's northland, immediately following the World War, hastened by the discovery of rich mines and a realization of the vast potential mineral wealth of the Pre-Cambrian shield, made a sudden demand on this Survey for topographical maps of hitherto unexplored country, and for surveys through this country to permit of the proper registration of all claims staked. The area to be surveyed is so vast that chained control surveys were out of the question, from considerations of speed as well as of cost. Hence, stadia traverses were run along the principal waterways for the dual purposes of providing a skeleton network of control surveys for the tying in of mineral claims and for ground control for the oblique aerial photographs from which the country was to be mapped.

In order to control these extensive stadia surveys, they were arranged in closed circuits wherever possible, and where this was not possible, astronomical observations for latitude and longitude were taken at intervals of 75 to 100 miles, and the stadia surveys were adjusted between observation points. The extent of these operations may be judged from the fact that in the six years immediately following the war, no less than 25 000 miles of stadia traverse were run by members of the Topographical Survey of Canada.

As regards the field work, it may be said that every precaution was taken to ensure freedom from gross errors both in directions and in distances, and many novel features were incorporated into the work with this end in view. Briefly, angle measurements, checked by frequent star observations for azimuth, are seldom more than 2′ or 3′ of arc in error, but the distances are subject to the inherent weakness of stadia measurements, so that as Mr. Rappleye rightly states, "the angle measures far surpass the distance measures in the matter of accuracy."

The field closing stipulated as being permissible between control points is $\frac{1}{400}$ of the surveyed distance, and these closing errors had to be distributed in some way. This led to an investigation of the method of adjustment best adapted to the conditions of the survey, yet not entailing too much office work. The tremendous amount of computations involved had a very important bearing on the method of adjustment to be adopted.

The investigation[3] was made by H. Parry, Mathematician of the Topographical Survey of Canada, who first examined the advantages and defects of the well-known Transit Rule[7] for balancing a closed survey. It was found that this rule produced appreciable changes in the accepted bearings of the courses, which were determined in the field with considerable accuracy. Consequently, an investigation was made to deduce a rule which would distribute the closing errors by correcting the lengths while leaving the bearings unchanged. This is exactly the same problem as that outlined by Mr. Rappleye, a least square adjustment of which was made with exactly the same results as obtained by him.

---

[6] Dept. of the Interior, Ottawa, Ont., Canada.

[7] "Theory and Practice of Surveying," by J. B. Johnson and L. S. Smith, Seventeenth Edition, 1912, pp. 235–236.

After some consideration, however, it was decided that this least square adjustment, while possessing the advantage of leaving the bearings unchanged, was not exactly suited to the field conditions under which Canadian Government surveys are run. It is based on the assumption that all the errors in the lengths are of the accidental class, while actually the lengths are also affected by systematic or constant errors which it is not possible to eliminate entirely. A study of these systematic errors shows that a certain proportion may be accounted for by variation in the stadia constant of the instrument with different climatic conditions. This is due partly to an actual variation in the distance apart of the stadia threads and partly to the consistent bending of the rays of light, especially the lower ray, under certain atmospheric conditions. While engaged on work of this nature in Canada, field surveyors of the Topographical Survey, are instructed to determine the stadia constant of their instrument from time to time during the course of the survey and where necessary, corrections are applied to the stadia lengths. Glass diaphragms are found to give a less variable stadia constant than spider threads. It is not possible, however, to correct for the systematic errors due to refraction and other minor causes.

In addition to these considerations, the least square adjustment, as applied to the extensive traverses of the Topographical Survey of Canada, would entail an enormous amount of computation, and this led to the selection of the much simpler Transit Rule. Incidentally, it might be mentioned that the least square adjustment necessitates corrections to the lengths which, in general, are greater than those given by the Transit Rule. This, of course, is obvious, since the Transit Rule besides changing the lengths, also changes the bearings up to as much as 8′, while the least square adjustment confines all the corrections to the lengths.

It will be seen, therefore, that the least square adjustment deduced by the author would not suit the field conditions encountered by the Topographical Survey of Canada. It can readily be understood, however, that entirely different conditions of field and office work might make it very desirable to adopt a least square adjustment, and the Surveying Profession is indebted to Mr. Rappleye for bringing this particular solution to its notice.

H. PARRY,[8] ESQ. (by letter).—In 1921 the writer made a study of the methods of adjusting stadia traverses in order to distribute the closing errors of the extensive stadia traverses of the Topographical Survey of Canada. As the Transit Rule[9] is by far the most widely known and used rule for adjusting and balancing traverse surveys, the following investigation into the effect of this rule on the bearings of the traverse will be of interest in connection with Mr. Rappleye's paper.

---

[8] Mathematician, The Topographical Survey of Canada, Ottawa, Ont., Canada.

[9] "Theory and Practice of Surveying," by J. B. Johnson and L. S. Smith, Seventeenth Edition, 1912, pp. 235–236, and other standard books on surveying.

The rule has been formally expressed by Johnson and Smith, as follows:

"As the arithmetical sum of all the latitudes is to any one latitude, so is the whole error in latitude to the correction to the corresponding latitude, each correction being so applied as to diminish the whole error in each case. Proceed similarly with the departures."

To evaluate the effect on the bearing of any particular course, let:

l = measured length of any particular course.
a = bearing of course.
L = arithmetical sum of the latitudes.
D = arithmetical sum of the departures.
F_L = error of closure in latitude.
F_D = error of closure in departure.
λ = correction to latitude of course considered.
μ = correction to departure of course considered.
δa = change in bearing of course considered caused by a correction of F_L to latitude and F_D to departure.

Stated algebraically, the Transit Rule for any particular course gives, for latitude:

$$\frac{\lambda}{l \cos a} = \frac{F_L}{L} \qquad (13)$$

and, for longitude:

$$\frac{\mu}{L \sin a} = \frac{F_D}{D} \qquad (14)$$

The corrected latitudes and departure of this course are $l \cos a + \lambda$ and $l \sin a + \mu$, respectively. Hence,

$$\tan (a + \delta a) = \frac{l \sin a + \mu}{l \cos a + \lambda} = \tan a \left\{ \frac{1 + \dfrac{\mu}{l \sin a}}{1 + \dfrac{\lambda}{l \cos a}} \right\}$$

$$= \tan a \left\{ 1 + \frac{F_D}{D} - \frac{F_L}{L} + \dots \right\} \qquad (15)$$

in which, terms of the second order are neglected.

Equation (15) is in the well-known form, $\tan x = \rho \tan y$, the solution of which is, $x - y = \dfrac{\rho - 1}{\rho + 1} \sin 2y + \dots$. Hence:

$$\delta a = \frac{\dfrac{F_D}{D} - \dfrac{F_L}{L}}{2 + \dfrac{F_D}{D} - \dfrac{F_L}{L}} \times \sin 2a + \dots \qquad (16)$$

or, neglecting terms of the second order:

$$\delta a = \frac{1}{2} \left(\frac{F_D}{D} - \frac{F_L}{L} \right) \sin 2a \qquad (17)$$

This is the change in bearing of a course of latitude, L, and departure, D, when the latitude is corrected by F_L and the departure by F_D.

In the stadia surveys of the Topographical Survey of Canada, the work is required to close with an error of less than 1 in 400. If this error of closure is due entirely to errors in the lengths, then the correction in latitude to any individual course must be less than 1 in 400 of the latitude of that course and the correction in departure must be less than 1 in 400 of the departure of that course.

This is not strictly true in practice because, in a long traverse running almost due north and south, in which the arithmetical sum of the departures is very small compared with the extent of the survey, the error of closing in departure due solely to small errors of direction might be of the order of 1 in 400. A similar condition would hold for a traverse running almost due east and west. However, in the general case the errors in direction are of a smaller order than the errors in lengths.

In the extreme case, assuming that latitudes are too long by 1 in 400, and departures are short by 1 in 400, or *vice versa,*

$$\frac{F_L}{L} = \frac{F_D}{D} = \pm \frac{1}{400}$$

and, by Equation (17),

$$\delta a = \pm \frac{1}{2} \left(\frac{1}{400} + \frac{1}{400}\right) \sin 2a = \pm \frac{1}{400} \sin 2a$$

or, in minutes of arc:

$$\delta a = 8.5 \sin 2a \ldots\ldots\ldots\ldots (18)$$

This is the change in bearing in the extreme case where latitudes and departures have their maximum allowable corrections of 1 in 400. It is a maximum for a course inclined at 45° to the meridian, in which case, $\delta a = \pm 8.5'$.

This introduction of errors into traverse bearings which have been accurately determined in the field is, then, the price that must be paid for ease and simplicity in the computations. It may be stated, however, that the work of the Topographical Survey of Canada is seldom affected by errors as large as 1 in 400, which are considered the maximum errors permitted without further investigation of the field work.

HOWARD S. RAPPLEYE, ASSOC. M. AM. SOC. C. E. (by letter).—The writer is indebted to Professor Underwood for a suggestion that is a decided improvement in the method of adjusting transit and stadia traverses, since his proposed method of weighting the various courses will undoubtedly result in better corrections. This method will also tend to remove the objection advanced in Professor Stewart's discussion.

The discussions by Mr. Peters and Mr. Parry present valuable information on the subject and serve to show that the treatment of the problem was checked, so to speak, before it was solved. It is superfluous, perhaps, to state that the writer had no knowledge of the work of the Canadian engineers in connection with this problem, until their discussions appeared.

The subject has been presented to the attention of the engineers of the country, much improved as a result of its discussion by others. It will be interesting to sit back and see what practical use is made of the method.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1768

CONSTRUCTION OF THE JAMES RIVER BRIDGE PROJECT[1]

BY R. C. WILSON,[2] ESQ., AND HERBERT B. POPE,[3] ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. WILSON T. BALLARD, WILLIAM WILLOUGHBY, AND R. C. WILSON AND HERBERT B. POPE

SYNOPSIS

This paper describes the James River Bridge Project and the more important details of its construction. This being a toll bridge and involving a large outlay, it was necessary to make a careful investigation of probable expenditures to determine the soundness of the investment. The item of cost most difficult to predetermine was that of the concrete pile foundation work, due to the soft clay bottom of the James River.

A description is given of the general plan of operation of the work from a main base where materials were assembled. The methods of casting, handling, and driving the large concrete piles are outlined, as well as other details of the work in the field. Actual load tests on piles which did not drive to refusal furnished interesting data in regard to their carrying capacity.

The James River Bridge Project (Fig. 1) is an important link in the Atlantic Coastal Highway System and makes it possible for a motorist to reach Norfolk, Va., from the north and continue south without the former long detours or troublesome ferries. The project consists of three bridges, totaling 5⅜ miles in length, and about 11 miles of concrete roads built to give access to the bridges from existing highways. The largest of these bridges crosses the James River just beyond the city limits of Newport News, Va., and is about 4½ miles long. The other two bridges carry the highway over Chuckatuck Inlet and the Nansemond River, respectively. The traveler will now find a splendid concrete roadway leaving the Richmond Highway just west of Newport News, spanning the James River, Chuckatuck Inlet, the Nansemond River, and connecting with the State Highway leading to Norfolk.

---

[1] Presented at the meeting of the Construction Division, New York, N. Y., January 17, 1929.

[2] Vice-Pres., Turner Constr. Co., New York, N. Y.

[3] Supt. of Constr., Turner Constr. Co., New York, N. Y.

Early Studies.—The preliminary work in connection with the project entailed first a very careful study on the part of the engineers as to the value of these bridges and connecting roads from an investment standpoint so that they might be certain that the revenue to be derived from tolls would represent a reasonable margin for profit beyond the cost of maintenance and the interest on the investment. A considerable amount of research work was necessary in order to determine even approximately the cost of the project. Surveys had to be made, borings and soundings had to be taken across the rivers on the proposed line of the bridges, and the cost of crossing private property with the right of way had to be ascertained.

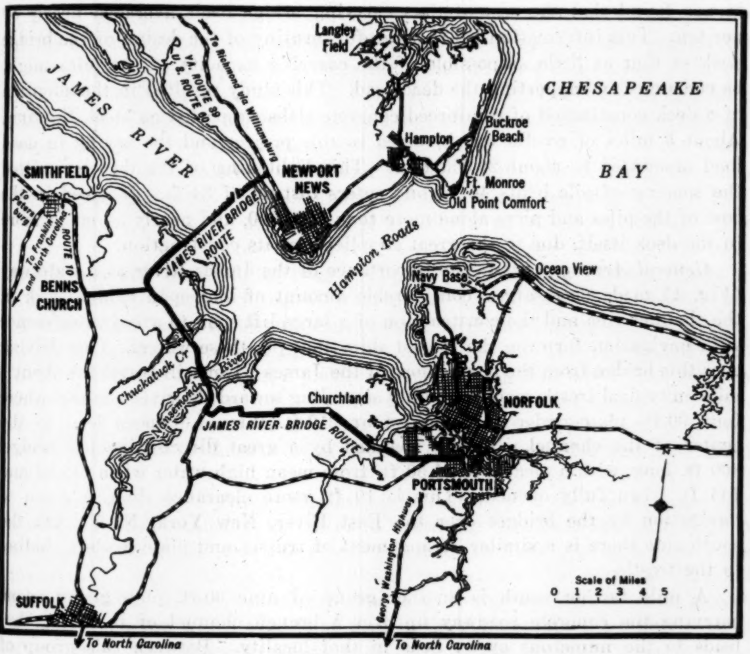

Langley Field
To Richmond via Williamsburg
VA. ROUTE 39
U.S. ROUTE 60
JAMES RIVER
CHESAPEAKE
BAY
Buckroe Beach
Hampton
SMITHFIELD
To Petersburg
To Franklin and North Carolina
ROUTE 10
NEWPORT NEWS
Ft. Monroe
Old Point Comfort
JAMES RIVER BRIDGE ROUTE
Hampton Roads
Navy Base
Ocean View
BENNS CHURCH
Chuckatuck Cr
Nansemond River
Churchland
NORFOLK
JAMES RIVER BRIDGE ROUTE
PORTSMOUTH
George Washington Highway
Scale of Miles
0 1 2 3 4 5
SUFFOLK
To North Carolina
To North Carolina

FIG. 1.—JAMES RIVER BRIDGE PROJECT WITH CONNECTIONS AND ENVIRONS.

It was assumed that pre-cast concrete piles would be used on this work, but as soon as the borings had been studied it was realized that some of these piles would have to be driven to unusual depths and that the resulting cost of the project might exceed the safe limit of investment. In order to obtain more accurate information, actual test piles similar to those finally used were cast and driven at intervals along the line of the James River Bridge. This work indicated that many of the piles would have to be of great length, but that a pile-supported trestle was feasible and would cost less than any other type of permanent bridge structure.

The largest piles indicated by the test were 115 ft. long and 24 in. square, and weighed slightly more than 35 tons each. Many of them did not drive to refusal, although penetrating as much as 90 ft. of soft clay. Actual tests had to be resorted to for determining what load such piles might safely carry. When they came to rest an enormous grip developed between the soft clay and the great area of pile surface. Thus, where existing formulas based on penetration under hammer impact indicated a carrying capacity of only 18 to 20 tons, the piles actually supported a load of 90 tons without settlement. The maximum bridge loads that the piles will ever be called upon to support will be 42 tons each.

When the estimate of making and driving these great piles was completed, it was found that the cost of thus carrying bridge loads averaged nearly $9 per ton. This information led to a careful scrutiny of the design of the bridge deck so that as little as possible of the carrying capacity of the piles might be consumed in supporting the dead load. This study resulted in the adoption of a deck constructed of reinforced concrete slabs supported on steel I-beams. About 5 miles of trestle was involved in this project and the saving in dead load amounted to about 26 000 tons. This lightening of the deck permitted the spacing of pile bents 44 ft. on centers instead of 34 ft. It saved in the cost of the piles and piers alone more than $300 000, and nearly as much more in the deck itself, due to the great simplicity of its construction.

General Arrangement.—The importance of the James River as a waterway (Fig. 1) made necessary a considerable amount of long-span bridge work in the channel area and the construction of a large lift span to avoid interference with navigation for even the largest ships that ply these waters. One driving over this bridge from the north shore of the James River will travel for about ¾ mile on typical trestle work gradually ascending toward the river channel where four 90-ft. plate-girder spans and four 210-ft. through trusses lead to the center of the channel. This is spanned by a great direct-drive lift bridge, 300 ft. long, with a clearance of 50 ft. from mean high water when closed and 145 ft. when fully opened. This is 10 ft. more clearance than is given to navigation by the bridges over the East River, New York, N. Y. On the south side there is a similar arrangement of trusses and plate girders leading to the trestle.

A mile farther south is another group of nine 90-ft. plate-girder spans carrying the concrete roadway up over a branch channel of the river that leads to the numerous oyster beds in that locality. Between this group of plate-girder spans and the south shore of the river is a fine stretch of typical trestle work more than 2 miles in length—straight and almost level. The two smaller bridges have the same type of trestle work as that over the James River. In each case the channel is spanned by a double-leaf bascule of the rolling type, 110 ft. in length, which gives ample clearance to navigation on these less important rivers.

Between the north end of the James River Bridge and the main highway running from Richmond, Va., to Newport News (Fig. 1) is an elaborate approach laid out around a plaza. At the south end of the same bridge for a distance of about a mile there was a difficult piece of roadway construction, through a swamp almost at tidal level. Here, the bottom was so soft that it had to be removed and sand pumped in from the river to form a suitable bed for the concrete roadway. About 8½ miles of concrete roads were constructed on the peninsula between the James and Chuckatuck, while between the Chuckatuck and the Nansemond about 1½ miles of concrete road had to be built, but these stretches are over higher land, where there was no trouble in finding proper foundation. Similar conditions exist to the south of Nansemond where about a mile of concrete roadway has been completed to connect with the existing highways leading to Portsmouth, Va., and Norfolk.

Locating Bridge Lines.—As part of the preliminary work on this project, extensive surveys were made. Base monuments had to be established by careful triangulation and permanent working points had to be set so that it would be possible to start construction at different locations along the line of the bridges without the probability of error of closure as each part of the work developed and approached the other. The initial triangulation work was done under the auspices of the U. S. Coast and Geodetic Survey which assigned engineers to establish the monuments designating the terminal points of the three bridges. No great problem was presented by the location of the roads, or even the two smaller bridges, and station points were established along the lines so that work could be started at any desired location.

The survey work on the James River was a more serious matter. The terminal points were nearly 24 000 ft. apart. Deep water intervened for most of the way and the visibility was apt to be poor. It was decided finally to erect pile-supported platforms at intervals along the line of the bridge and on these rigid bases to locate accurate transit points that would be used as reference marks in starting any section of the work. The instrument point in the center was located on a platform supported on four wood piles securely wrapped and stapled together. The working platform surrounding the instrument point and independent of it was also built on wood piles and permitted mooring of boats to the platforms without danger of moving the instrument point.

Material Testing.—A concrete testing laboratory with modern and complete equipment was established at the main operating base in Newport News. A screen analysis of the sand and gravel was made as these materials were being towed to the site of the work, and exact instructions were given at frequent intervals to the different mixing plants as to the proportions of sand, gravel, cement, and water which were to be used in the concrete for the different parts of the work. Specimen cylinders were made of the concrete actually being used each day and these were broken at 7, 14, and 28 days, so that there might be an exact control of the strengths of concrete specified without any waste of cement.

Other test cylinders, cured under various weather conditions, were broken at from 2 to 10 days, and the information thus gained as to the early strengths of concrete was used as guides in determining the safe time for removing the forms. This led to economy in form building by not having any of the centering left in place for unnecessary lengths of time. A 1 : 1½ : 3 mix was used in the concrete piles and for all cap and pier work to Elevation + 5.0. All other concrete was designed for a 1 : 6 mix.

Working Base.—In the construction of this project the problem of first importance was the establishment of a main operating base. For this, certain qualifications were essential, such as:

1.—Deep-water wharfage.
2.—Rail connection.
3.—Electric power; fresh water supply.
4.—Harbor for refuge of floating equipment.
5.—Sufficient area for storage of materials, pile-casting, and storage yard, carpenter shops, offices, and room for fabrication and storage of reinforcement, and many other items.
6.—Reasonable accessibility to the different parts of such an extensive work.

A survey of available sites, fortunately revealed one that was ideal for all requirements. Accordingly, a lease was taken of the major part of one of the shipyards that had sprung into being during the World War. This property was situated within the city limits of Newport News and fronted for a distance of 700 ft. on Hampton Roads, with a wharfage of 1 700 ft. along a small boat harbor where floating equipment could be tied up. All this dockage was required because when the many craft had to seek shelter they were moored two and three deep along the bulkhead.

As shown in Fig. 2 the principal area was devoted to the casting and curing (30 days) of the 2 980 concrete piles. Thus, with the sections required for the fabrication of the reinforcement and the casting it was necessary to set aside for the pile yard a space, 1 100 ft. long and 125 ft. wide. This was served by a gantry crane which ran out over a slip on Hampton Roads where the piles were loaded on barges for transportation to the drivers.

Other principal parts of this yard were devoted to the storage of the heavy I-beams used in the deck construction, to saw-mills, to form-making yards, to storage of reinforcing steel and its fabrication, to space for building special barges, etc. Along the boat harbor, space was reserved for an emergency storage of sand and gravel. Fortunately, a marine railway was still in operation along the west side of this yard and hardly a week went by but one or another of the many pieces of floating equipment found its way to this railway for repairs.

Pile-Casting and Handling.—The pile-casting yard proper was constructed on a forest of 2 by 4-in. stakes driven into the ground. These stakes were spaced 18 in. apart, running lengthwise of a supporting 2 by 4-in. joist, and the joists themselves were spaced 15 in. apart. On top of the joists, ⅞-in. sheeting was laid for the width of the pile. The side forms for the pile were held at the bottom by a 2 by 4-in. cleat nailed to the joists; at the top they were braced and plumbed by a specially constructed removable cap member. Later, this was taken off to permit the side forms to spring apart and thus allow the lifting of the pile. The casting yard had a capacity of 140 piles, the lengths being adjusted by putting the end stoppers in the forms at any desired position.

Piles as long as 115 ft. could be poured on this platform. Longitudinal steel for the piles came in full lengths—117 ft. for the longest units.

The mixing plant for the casting yard consisted of two 1-yd. mixers which were fed from a large charging hopper holding approximately 400 yd. This hopper was loaded by means of a stiff-leg derrick which was located on the bulkhead in such a position that material could be charged directly into the hopper, either from the barges on which it arrived, or taken from a storage pile located under the swing of the boom. Aggregate entering the mixer was measured by batch hoppers and the water content was gauged accurately by means of an approved measuring tank. Cement was brought to the mixing plant from the cement house by a belt conveyor.

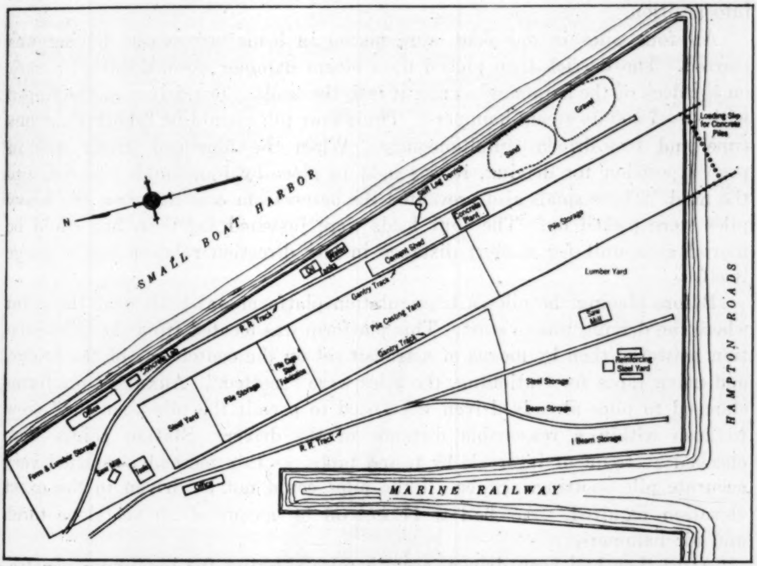

N
SMALL BOAT HARBOR
Oil
Water
Tanks
Cement Shed
Stiff Leg Derrick
Concrete Plant
Sand
Gravel
Loading Slip for Concrete Piles
Pile Storage
Lumber Yard
Gantry Track
Pile Casting Yard
Gantry Track
R. R. Track
Concrete Lab
Office
Steel Yard
Pile Storage
Pile Cage Steel Fabrication
Form & Lumber Storage
Saw Mill
Tools
Office
R. R. Track
Saw Mill
Reinforcing Steel Yard
Steel Storage
Beam Storage
I Beam Storage
HAMPTON ROADS
MARINE RAILWAY

FIG. 2.—ARRANGEMENT OF OPERATING BASE, JAMES RIVER BRIDGE PROJECT, AT NEWPORT NEWS, VA.

After the piles had been poured for a period of approximately 7 days they were lifted from the forms by the gantry crane spanning the casting platform, using for this purpose, heavy T-bolts with washers, cast into the piles. Either three or four bolts were used, depending on the length of the pile. The loads on the T-bolts were balanced by blocks on an equalizing beam slung under the gantry. After lifting, the pile was carried to the curing yard, also under the crane (Fig. 3). There it was thoroughly inspected for cracks and flaws and was cured for a period of 21 days, or longer, by means of water jets. This yard had a capacity of approximately 500 piles. As piles were required they were picked up by the gantry crane which, running out past the bulkhead, was able to place them directly on a barge for transportation to the drivers.

Pile-Driving.—Shallow water near the shore on all three bridges necessitated the dredging of a channel 180 ft. wide and 6½ ft. deep along the line of the bridges from deep water to shore. These channels permitted the efficient use of the heavy floating equipment necessary for handling piles and other loads. The equipment used for placing the piles consisted of two large floating drivers. One was of the ordinary single-lead type; the other was called the "four-lead" driver (Fig. 4), and carried a derrick capable of handling the 115-ft. piles. This derrick picked up the pile from the transportation barge and placed it in the leads, using a rather complicated sling, with three or four points of suspension along the pile depending on its length, the loads being compensated so that no excessive strain was placed on it during the entire operation. When the pile was hanging directly plumbed, it was placed in the leads and lowered into position.

All four piles in one bent were placed in leads before the driving was started. The derrick then picked up a steam hammer assembly from a rack on the deck of the barge and swung it into the leads. The driver was equipped with two 7 500-lb. steam hammers. Thus, four piles could be "spotted" at one time and two driven simultaneously. When the four-lead driver was in proper position for driving, it was held in place by four spuds lowered into the mud. These spuds also prevented the barge from canting when the heavy piles were picked up. The four leads were fastened together, but could be moved as a unit for a short distance in any direction relative to the barge itself.

Before placing the piles a triangulation platform was built near the point where the driving was to start. This platform was located from the triangulation network; then by means of a transit set on the center line of the bridge, and using tapes for stationing, the piles were "spotted." Auxiliary platforms clamped to piles already driven were used to permit the pile-"spotting" crew to keep within a reasonable distance of the driver. Station points were checked at frequent intervals by tested tapes, as this method permitted very accurate pile spotting. Since all the piles could not be driven to the exact elevation required, some had to be cut off by means of an acetylene torch and air hammers.

After they had been driven, certain piles showing the maximum penetration per blow were selected for testing. A water tank was placed on top of an I-beam frame slipped over the top of the pile (Fig. 5), and was filled with water. Readings were made for various loads until a maximum of 90 tons was reached. Observations were taken for settlement under each increment of load. After the maximum was reached, the tank was emptied and readings were taken to determine how much, if any, the pile itself had been compressed or how much it regained under decreased load. It was found that piles which had a comparatively large penetration per blow while being driven showed only about ⅛-in. settlement by the maximum load of 90 tons.

Floating Concrete Plant.—Each of the two concreting plants was built on a special barge, 36 ft. wide by 91 ft. long, with a maximum draft of 7 ft. A stiff-leg derrick was placed at the rear for moving sand and gravel from scows moored on either side, to the bin over the 1-yd. concrete mixer; a 65-ft.

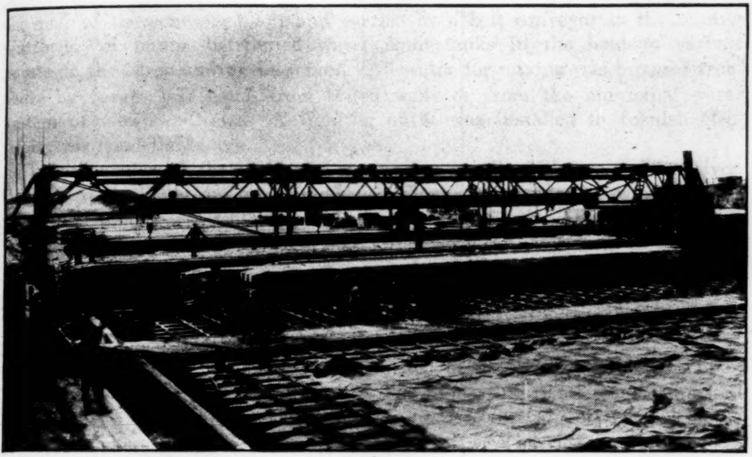


FIG. 3.—PILE-CASTING YARD, SHOWING GANTRY PICKING UP 115-FOOT PILE READY TO LOWER IT INTO STORAGE.

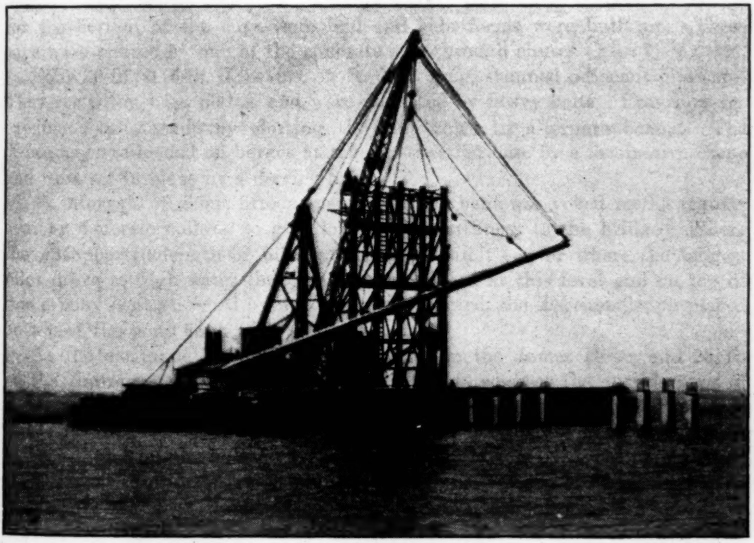


FIG. 4.—FOUR-LEAD PILE-DRIVER.

tower directly in front of the mixer hoisted the concrete to a distributing hopper on the face of the tower. The cement was delivered by barges at the rear of the concrete plant and carried by a belt conveyor to the mixing platform. A pump distributed water from tanks in the hold to various points on the barge and to the mixer. All water for mixing was brought from shore by barges. It came from tested wells or from the municipal water system of Newport News. A lighting outfit was installed to furnish electricity for flood lights, etc.

In the center of the barge was an anchor-handling device, power-driven through reduction gears to "nigger heads" at either side, by which means the plant could work itself along the bridge as desired. All this equipment was gas-driven and required about 250 h.p. in seven units of from 10 to 80 h.p. Wood-pile dolphins were driven at intervals of perhaps 1 000 ft. parallel to and about 500 ft. from the center line of the bridge. These were used for mooring the various equipment at night and also served as a point of attachment for the stern lines of the concrete rigs.

Trestle Deck.—The typical deck consisted of longitudinal I-beams upon which a 9-in. concrete slab was poured. The supporting trestle bent comprised four piles, 7 ft. on centers, capped by a concrete beam (Fig. 6) bonded to the pile by the pile steel itself. Forms for these caps were carried on timber yokes around each pile; on top of these yokes were laid two 6 by 10-in. timbers the full length of the bent. These timbers were likewise clamped by bolts to the piles and not only afforded a greater resistance to slipping under the concrete load, but also pulled the piles into line. On top of these timbers the forms for the bottom of the caps were laid and side forms were built up. These caps were poured by one of the concrete rigs through chutes (Fig. 7).

Four 30-in., 115-lb. I-beams, 43 ft. 9 in. long, spanned adjacent pile caps. They rested on base plates, and were anchored by heavy bolts. Provision for expansion was made by slotting the bolt holes in alternate beams. The I-beams were loaded on barges at the construction base by a locomotive crane and were set in place by a derrick boat.

At intervals of about fifteen spans, an extra bent was yoked to the regular bent by concrete collars, to give longitudinal stiffness to the bridge. Where the unsupported length of piles exceeded the limit set, or where the longest piles drove to high-water line, the cap was placed at this level and on top of this a pony bent stiffened by web walls was erected, the I-beams being placed on top of the pony bent.

As the clearance between curbs is 22 ft. on the James River and 23 ft. on the Nansemond River and Chuckatuck Creek, whereas the outside rows of I-beams are only 21 ft. on center, a small cantilever slab projected on each side. To support the forms for this, 4 by 8-in. outriggers were used; these, in turn, hung on the bottom flange of the outside row of I-beams and used the adjacent interior beams as points of reaction. On top of these outriggers, to support the floor forms, posts were placed, which also gave a means of bracing the outside of the curb forms (Fig. 8). For the interior panel forms between the I-beams themselves, 4 by 4-in. legs, with 2 by 6-in. headers, rested directly on the bottom flanges of the beams. Being of exact height they did not require wedging. On top of both sets of jacks were laid panels constructed of ⅞-in. tongue and groove material (Fig. 10). Because the curb was poured monolithic with the slab itself, some means for supporting the inside vertical forms was necessary. A steel frame was used supported from the outside form of the curb by angle brackets (Fig. 8). This obviated the necessity of having any wood or concrete blocks in the deck slab itself for the support of the form. The deck was smoothed by a finishing machine running on a track which was supported on chair-castings resting directly on the outside I-beams. After the finishing machine had passed, the track and castings were removed, and this section of the deck was hand-finished.

Reinforcing steel for the deck slab consisted of ⅝-in. bars running across the I-beams and raised over the points of support (Fig. 8). Concrete was poured directly from the barge as it pulled itself along. This method of operation obviated the necessity of wheeling the concrete, and thus permitted the finishing machine to follow closely behind the actual pouring. The slab was checked for bumps by light straight-edges, was given a float finish, and, finally, the float marks were rubbed out by dragging burlap over the surface. The deck was cured by being kept wet for fourteen days, and in warm weather was further protected against the sun by a burlap covering for several days.

Bridge Structures.—The piers supporting the trusses were in about 40 ft. of water and were supported on wood piles driven practically flush with the river bottom. On top of this piling were placed bell-shaped steel cylinders, two for each pier, extending to a point 4 ft. above mean low water. These cylinders flare from a diameter of 23 ft. at the bottom to one of 10 ft. at the water surface (Fig. 9). The piers comprised two cylinders with a connecting web-wall from Elevation + 4 to the bottom of the structural steel work. The foundation piers for the James River Bridge lift were 28 by 52 ft. at the river bottom, supported on wood piles, and were built inside single-wall, steel, sheet-pile coffer-dams. A concrete seal 15 ft. thick was placed inside these coffer-dams by tremie as in the case of the cylinder piers, after which they were unwatered and the piers themselves were built without difficulty. The supporting piers for the 90-ft. plate-girder spans were built on concrete piles capped at water level. Fig. 10 gives an idea of the extent and appearance of this entire structure.

The structural steel in the long spans was fabricated on falsework by means of a traveler, and each tower for the lift span was raised by a derrick designed especially for the work. The floor system of these truss spans was very simple and consisted of transverse floor-beams and four lines of stringers on which the concrete was poured, using forms similar to those for the trestle deck. The floor of the lift span had stringers placed much closer together, over which 2 by 8-in. creosoted planks were laid on edge. The wearing surface consisted of a 1½-in. layer of asphalt, fabricated in strips; these were laid very much like a plank flooring, the joints being poured with asphalt. All steel work was painted with one shop coat of red lead and two field coats of carbon paint, the final coat being black.

Each channel was protected by a fender 200 ft. long with 50-ft. flares. It was constructed of creosoted piling spaced 8 ft. on centers, capped by a 12

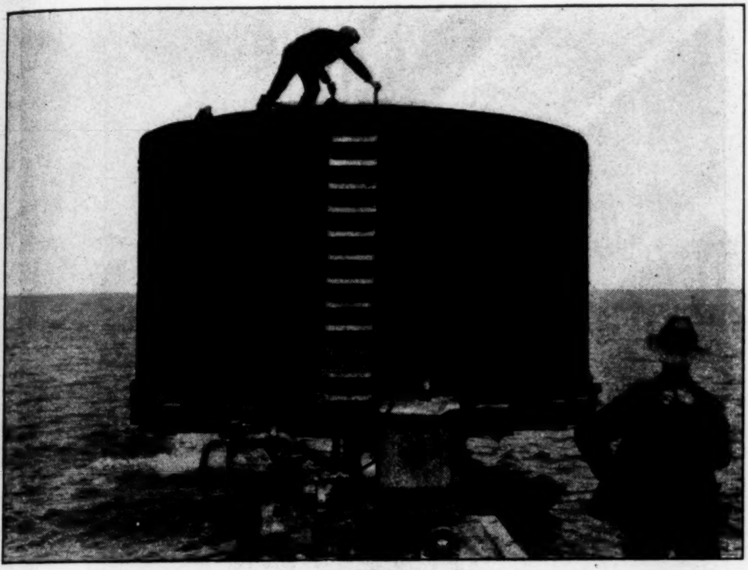


FIG. 5.—LOAD TEST FOR PILE.

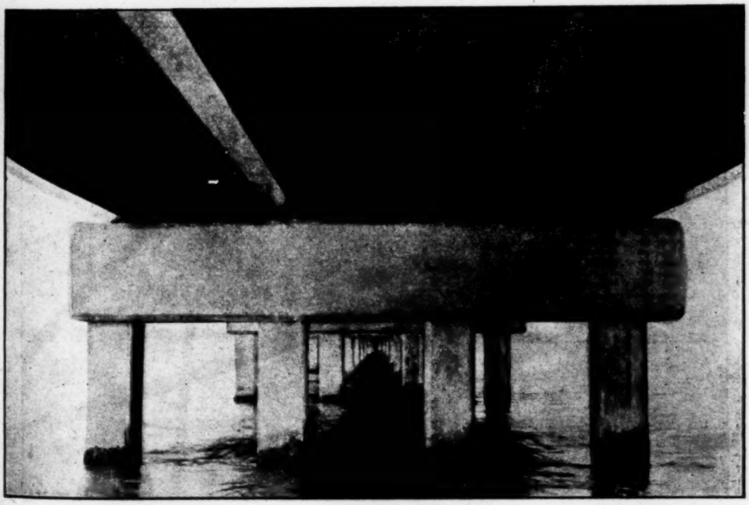


FIG. 6.—UNDER VIEW OF JAMES RIVER BRIDGE, SHOWING CONCRETE TRESTLE BENT CARRYING STEEL STRINGERS.

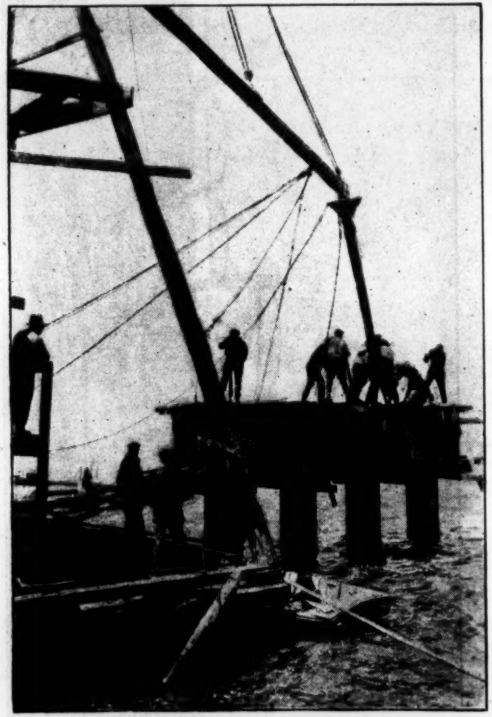


FIG. 7.—CONCRETING PILE CAPS.

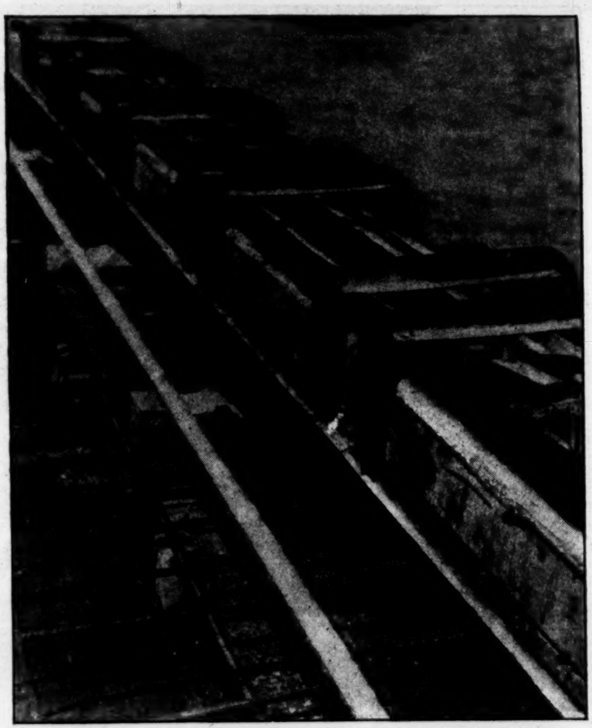


FIG. 8.—CURB FORMS WITH SLAB REINFORCEMENT.

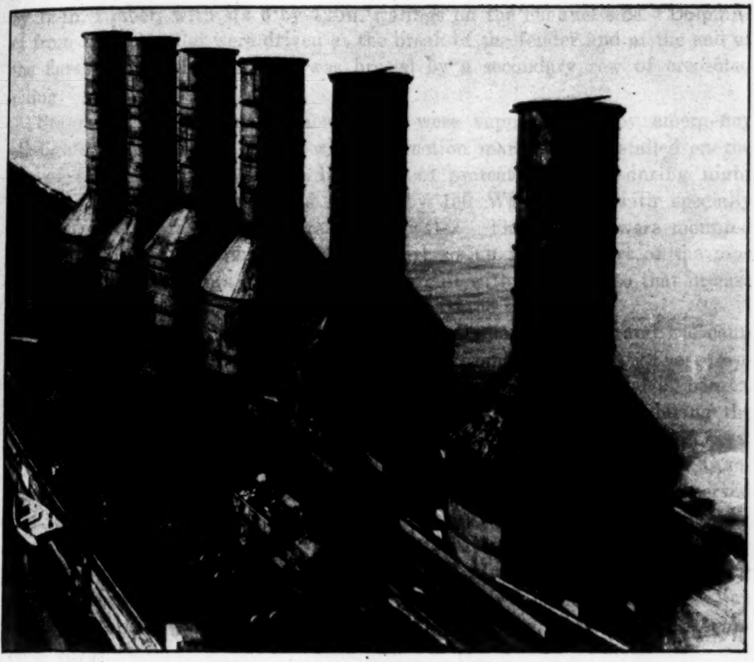


FIG. 9.—FORMS FOR CYLINDER PIERS.

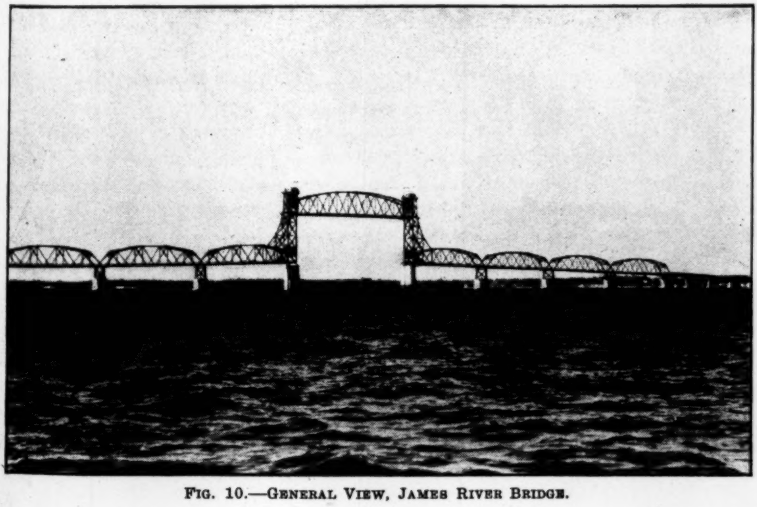


FIG. 10.—GENERAL VIEW, JAMES RIVER BRIDGE.

by 12-in. timber, with six 6 by 12-in. walings on the channel side. Dolphins of from 13 to 21 piles were driven at the break of the fender and at the end of the flares. The whole system was braced by a secondary row of creosoted piling.

Standard electrical navigation lights were supplemented by emergency oil lights. An aviation beacon with a direction marker was installed on the top of the north tower at the lift span to protect airplanes during night flying. The bridge proper was lighted by 150 Watt lamps with specially designed shields to prevent glare on the water. These lights were mounted on 18-ft. poles, approximately 260 ft. apart, as an integral part of the pipe railing. They were in a 3 300-volt series circuit with provision so that in case one bulb became "dead", the others would continue to burn.

Completion.—The first pile was driven on January 2, 1928, and the complete job was turned over to the owners on November 17, 1928. The equipment used in the river work consisted of more than sixty power boats, barges, dredges, and pile-drivers of various sizes. There were employed during the period of maximum activity more than 1 000 men on the whole work.

This project, costing about $6 000 000, was financed by Paine-Webber Company, of Boston, Mass. The J. E. Greiner Company, of Baltimore, Md., served as Engineers. The Turner Construction Company of New York was the general contractor and associated with it were the Raymond Concrete Pile Company, New York, concrete piles; Sanford and Brooks, Baltimore, dredging and deep-water piers; Virginia Bridge and Iron Company, Roanoke, Va., structural bridge work; and Roberts Paving Company, Salisbury, Md., concrete roads.

DISCUSSION

Wilson T. Ballard,[4] M. Am. Soc. C. E.—Two points mentioned briefly in this paper are worthy of further discussion, namely, preliminary studies of the subsurface conditions along the sites of the three bridges, and actual load tests made upon reinforced concrete bearing piles driven in the trestle portions of the Nansemond River Bridge and the James River Bridge.

Subsurface explorations of the bridge sites were made by borings, by driving timber test piles, and by driving reinforced concrete piles similar in every respect to the permanent piles in the structure. The borings and the behavior of timber test piles during driving furnished information upon which was based the tentative decision as to the general type and layout of the bridges, and the preliminary estimate of cost of the entire project. However, it did not furnish sufficiently reliable information upon which to base a final schedule of the lengths to be used in casting the concrete piles nor in determining the location of piles of various lengths in the bridges. For example, concrete pile lengths indicated by the behavior of timber piles driven in Chuckatuck Creek were from 15 to 20 ft. too long. As a further example, reinforced concrete piles could not be driven to more than one-half the desired penetration at the sites of the lift-span piers of the James River Bridge, whereas timber piles were driven successfully to the desired penetration of from 40 to 45 ft. through a hard blue marl. Had the material through which these piles were driven, been sand, it would have been possible to sink reinforced concrete piles to the desired penetration by the use of a jet.

The driving of test piles of the same size and kind as those designed for the finished structure furnished very reliable data for scheduling the pile lengths to be cast for various locations. Therefore, such piles were spotted at various points ahead of the location where driving of permanent bearing piles was in progress, and the casting schedule was governed by information thus obtained. As a result, there remained only one unused pile after all the permanent bearing piles had been driven.

In view of the great cost of making subsurface exploration of bridge sites similar in length and character to those of the three bridges of the James River Project, it is believed to be wise economy to confine the investigation to the driving of test piles exactly similar in every respect to those intended for use in the permanent structure. The pile-driver used on the test piles should be of the same type and weight as that to be used in driving the permanent piles. The information thus obtained may be supplemented by sufficient test borings to determine the character and depth of the underlying soil strata.

The load tests made on test piles and permanent bearing piles brought out interesting facts which add confirmation to the belief that the bearing value of large concrete piles driven in various types of soft material cannot be determined by the pile-driving formulas in general use. As stated by the authors, each test was made by mounting a steel tank upon the pile and filling it with water. The load was applied rapidly until it reached about 30 tons and thereafter more slowly. Observations of settlement were taken during and after loading. Of the eight tests made on permanent piles the time that elapsed during the application of the load was from 1½ to 3 days. The maximum test load was allowed to remain in place for 2 to 5 days, except in the case of two piles in the Nansemond River, from which it was removed after one day.

---

[4] Vice-Pres. The J. E. Greiner Co., Baltimore, Md.

During the summer of 1927, several reinforced concrete test piles were driven along the site adopted for the James River Bridge, in that length of the river where borings and soundings indicated that greatest pile lengths would be required. This part of the river included a distance of about 6 000 ft. along the center line of the bridge and lay south of the present main channel. It may have been a deep channel in times past which was gradually deepened with the general subsidence of the eastern seaboard and filled up with clay and silt brought down by the river. These test piles varied in length from 90 to 114 ft. and were 24 in. square in cross-section. They were driven by a No. 1 Vulcan single-acting hammer, with a ram weighing 6 500 lb. and with a 3-ft. stroke. None of them could be driven to refusal in the soft bottom. Four were subjected to test loads sufficient to produce failure and the results were compared with calculated bearing values found by the use of the following pile-driving formulas:

Greiner formula:

$$P = \frac{1.07\,w\,h}{S + 0.1} \quad \cdots\cdots\cdots \quad (1)$$

Engineering News-Record formula:

$$P = \frac{2\,w\,h}{S + 0.1} \quad \cdots\cdots\cdots \quad (2)$$

Wellington formula:

$$P = \frac{2\,w\,h}{S + 0.1\frac{W}{w}} \quad \cdots\cdots\cdots \quad (3)$$

wherein:

P = allowable design lead on pile, in pounds.
h = stroke of hammer, in feet.
S = penetration of pile, in inches, under final blow.
w = weight of striking parts of hammer.
W = weight of pile, in pounds.

Table 1 shows the results of the test compared with the calculated bearing values of the piles found by the use of Equations (1), (2), and (3).

These results showed the futility of attempting to govern the driving of the permanent piles by formula. As a result of these preliminary tests, it was decided to attempt to drive the permanent bearing-piles to a final set of not more than ¼ in. per final blow under the No. 0 Vulcan hammer (weight of ram, 7 500 lb., and stroke of 3.5 ft.).

When the driving of the permanent piles was commenced in the Nansemond River in January, 1928, it was soon evident that this could not be done with piles of reasonable length. It was then decided to continue driving, to observe and record carefully the behavior of the piles, and to subject each of those that offered the least resistance to driving to an actual test load equal to twice the total load the pile would be required to carry in the finished bridge. Four of those driven in the Nansemond River were selected for test; two were 65 ft. long by 18 in. square, one, 70 ft. long by 18 in. square, and one, 95 ft. long by 24 in. square. Boring data obtained along the site of the bridge indicated that the three 18-in. piles penetrated from 10 to 15 ft. of soft mud and 30 to 35 ft. of soft blue clay or soft peat, while the 24 by 24-in. pile penetrated 15 ft. of soft black mud, 5 ft. of soft peat, 30 ft. of soft blue clay, 23 ft. of silty peat, and 15 ft. of hard, fine sand. However, the driving record of the 95-ft. pile would indicate that it probably did not penetrate much, if any, sand as it drove with comparative ease until its tip was 88.7 ft. below mean low water. The number of blows required to produce the final foot of penetration of each of these four piles is shown in Table 2 (Piles *A, B, C,* and *D*), together with the results of the load tests.

TABLE 1.—COMPARISON OF RESULTS OF ACTUAL LOAD TESTS WITH BEARING VALUES COMPUTED BY THE USE OF FORMULAS.

Pile No.	Equation.	Penetration, S, in inches, under final blow.	Computed bearing value, P, in tons.	Maximum test load that caused no settlement, in tons.	* Load that produced failure, in tons.	Factor of safety based upon bearing value, computed by equations.
1	(1)	0.5	17.4	20	85	4.9
	(2)		32.5	20	85	2.6
	(3)		14.7	20	85	5.8
2	(1)	0.095	53.5	30	121	2.3
	(2)		100.0	30	121	1.2
	(3)		17.4	30	121	7.0
3	(1)	0.25	29.8	30	85	2.9
	(2)		55.7	30	85	1.5
	(3)		17.0	30	85	5.0
4	(1)	0.226	32.1	30	103	3.2
	(2)		62	30	103	1.7
	(3)		15.3	30	103	6.7

* The load that produced failure is defined as that which produced rapid settlement that continued until part or all of the test load was removed.

After piles in the bent that included Pile *A* had been in place for 24 hours, the driver was placed on a pile adjacent to Pile *A* and an attempt was made to drive it farther. Fifty blows of the hammer drove the pile ¾ in. After a brief interval, twelve more blows produced no further penetration.

After the removal of the test load, Pile *D* (Table 2) recovered $\frac{5}{16}$ in., indicating that the permanent settlement caused by the load was ⅛ in. The three 18-in. piles each recovered approximately ⅛ in. on removal of the test load.

As a result of the tests of these four piles confidence was established in the fact that the load-carrying capacity of concrete piles that could be driven to a set of from ½ in. to ¾ in. through strata such as were encountered in the bottom of the Nansemond River, would be safe for the loads to be imposed by the bridge in service.

During the driving of piles for the James River Bridge, several could not be driven to a final set of ½ in. to ¾ in. with pile lengths of 115 ft. It was decided, therefore, to make load tests on four of these piles which offered the least resistance to driving under the No. 0 Vulcan hammer, as used in Nansemond River. The four selected were each 24 in. square and 115 ft. long. According to borings made along the bridge site, each of these piles penetrated about 30 ft. of water, 50 ft. of stiff blue clay, from 5 to 10 ft. of soft blue clay, and from 20 to 25 ft. of sand. The behavior of these four piles during driving, however, would indicate that it was very doubtful if they penetrated any sand, as the number of blows of the driver for the final foot of penetration did not exceed ten for any of them. As the bottoms of the Nansemond River and James River, particularly the latter, vary greatly in character within short distances measured along the center lines of the bridges, and since they have at frequent intervals deep pockets of soft material, it seems quite likely that such pockets may have occurred at the locations of the four piles in the Nansemond and the four piles in the James River that were tested, without being indicated by the boring data.

TABLE 2.—RESULTS OF LOAD TESTS.

Pile.	Length, in feet.	Size, in inches square.	Number of blows per final foot.	Maximum design load, in tons.	Test load, in tons.	Total settlement under test load, in inches.
A	70	18	24	34.4	68.8	9/64
B	65	18	18	34.4	68.8	1/4
C	65	18	32	34.4	68.8	1/4
D	95	24	28	42.0	94.0	7/16
E	115	24	9	40	80	1/8
F	115	24	5	40	80	3/16
G	115	24	10	40	80	17/32
H	115	24	8	40	87.7	6

Table 2 shows a summary of the driving data and load-test results of the four piles (*E, F, G,* and *H*) in the James River Bridge. The test on Pile *H* is considered unfair because a severe storm with high wind prevailed. Heavy waves beat against the test tank and this probably caused the pile to loosen and settle as noted; settlement stopped when the filled test tank came to rest on adjacent piles. It should be stated that during the first three days of the storm, the pile settled only 1½ in. under a test load of 53 tons.

As a result of these tests it was decided that piles of lengths equal to those tested, and which behaved in a similar manner, could be relied upon to carry safely the maximum design load of 40 tons per pile. The three bridges have been in service since 1928 and there is no apparent evidence of pile settlement in any of them.

It is felt that the foregoing facts show that, when it is intended to support structures on piles, it is wise economy to limit subsurface exploration of sites similar to those described herein, to the driving of test piles identical in every respect to those intended for use in the permanent structure. Consideration of the results of the load tests, adds confirmation to the belief that, in pile-driving of the nature described, the various formulas in general use cannot be relied upon to govern the driving, nor to determine the economic safe carrying capacity of the piles. Load tests should be made for this purpose.

WILLIAM WILLOUGHBY,[5] ASSOC. M. AM. SOC. C. E. (by letter).—The writer intends to confine his remarks principally to some amplification of that part of this paper pertaining to the structures over the navigable areas of the James River project.

To meet the requirements of navigation it was necessary to provide four openings, one in each of the bridges over the Nansemond River and Chuckatuck Creek and two in the bridge across the James River. (See Fig. 1.)

Bridges Over Nansemond River and Chuckatuck Creek.—The channels of the Nansemond River and Chuckatuck Creek were spanned with double-leaf deck-plate girder bascules of the rolling type, 110 ft., center to center of bearings. Each leaf was supported on a U-shaped pier founded on fifty-four 2-ft. square concrete piles, 80 to 85 ft. long.

The bridges are electrically operated and are equipped with solenoid service and emergency brakes. Limit switches cut out at the "open", "nearly open", "closed", and "nearly closed" positions, and an indicating light shows the interleave position of the shear locks. The bridge-operating circuits are connected in series with the warning devices and safety gates so that the bridge cannot be opened until warning is given and the gates are closed; nor can the gates again be opened until the bridge is entirely closed. The time required for operating the span is 1 min. Emergency operation by hand requires 12 min.

James River Bridge.—Nine 90-ft. through plate-girder spans across the south channel of the James River provide a horizontal clearance of 80 ft. and a vertical clearance of 25 ft. above mean high water. These spans are supported on rectangular piers on concrete piles. This opening is approximately 1 mile south of the main channel and provides access for small fishing boats to the large oyster beds on each side of the bridge.

The structure across the main channel of the James River consists of a 300-ft. vertical lift span. On each side of this span there are four 210-ft. truss spans and four 90-ft. through plate-girder spans (see Fig. 10). The truss spans are through bridges of the Warren type. The towers of the vertical lift span are connected to the adjacent 210-ft. spans in such a way that the vertical leg of the towers rests on the channel piers and the back leg frames into the span at the second panel point. This structure provides a minimum vertical clearance of 35 ft. for a distance of 1 980 ft. The lift span over the channel provides a horizontal clearance of 250 ft., a vertical clearance of 50 ft. above mean high water when closed, and a clearance of 145 ft. when open.

The piers adjacent to the channel are rectangular in shape; the base is 28 ft. wide and 52 ft. long, supported on 190 wooden piles, each 50 ft. in length. The piers under the truss spans consist of two cylinders, connected by a web wall above Elevation + 4.0. These cylinders are 10 ft. in diameter at the water-line and are flared to 23 ft. at the base. Each cylinder is supported on forty-one wooden piles varying from 40 to 50 ft. in length at all but three piers, where the piles are from 70 to 80 ft. long. The plate-girder spans rest on rectangular piers supported by twelve concrete piles 2 ft. square.

---

[5] Res. Engr., The J. E. Greiner Co., Baltimore, Md.

The entire site of the piers was excavated to a depth of from 6 to 10 ft. below the bed of the river, or from 30 to 40 ft. below mean high water. The piles were then driven until their tops were approximately level with the original river bed, or to refusal. They were then cut off by divers at about 8 ft. above the bottom of the excavation. In the cylinders the piles were driven in three concentric circles the radii of which were 3 ft. 3 in., 6 ft. 6 in., and 8 ft. 9 in. The center of the piles in the outside circle was thus only 2 ft. 9 in. from the location of the steel cylinder. In the channel piers the piles were spaced in nineteen rows of ten piles each and came within 1 ft. 6 in. of the line along which the sheet-piling was to be driven. It will be noted that, due to the small amount of clearance between the piles and cylinders, or sheet-piling, it was necessary to use extreme care in locating and driving the piles so that the cylinders would go down over the pile clusters, and the sheet-piling (in the case of the channel piers) could be driven without interference.

A pile dock was built near the site of the piers from which measurements could be made for "spotting" the piles; these docks were located by triangulation because the channel is a mile from the nearest shore. Except in the case of the three piers in which 70 to 80-ft. piles were used, the piles were driven through a steel tube 26 in. in diameter. The tube was provided with guides that fitted the leads of the pile-driver and was so constructed that it could be raised or lowered. The pile-driver was located at the correct point, and the tube was lowered until it rested on the bottom of the excavated hole. It was long enough to reach from this point to about 5 ft. above the surface. The pile was then placed in the tube and driven by means of a follower until its top was at the desired elevation. The required length had been previously determined by driving test piles at the site of each pier. In the case of the long piles sufficient penetration was secured, before it was necessary to use the follower, to guide them down to their correct positions. The steel cylinders were then lowered over the pile clusters, guided by divers or, in the case of the channel piers, coffer-dams were constructed. In no case did the piles interfere with these operations and this good fortune was due to the care with which they were driven. A submarine seal was then placed by means of a "tremie" pipe so that the piles were embedded. In filling the cylinders, the concrete was brought to within about 15 ft. of the surface in one continuous operation. A 15-ft. seal was placed in the channel piers. The coffer-dams and cylinders were then pumped out, and the remainder of the pier was constructed in the open. The coffer-dams of the channel piers were single-wall, steel, sheet-pile dams, securely braced, and were unwatered to a depth of 30 ft. The dams were very tight and were kept dry with very little pumping.

At the site of the cylinder piers, it was necessary to construct heavy docks to which the steel cylinders were secured. This prevented the cylinders from rising as the concrete, in its fluid state, rose in the flared section.

The vertical lift span is of the direct-drive type, thus eliminating the heavy machinery required for its operation from the span. On top of each tower is a house in which is located the machinery for the operation of the span. The machinery consists of a gear train on each tower, connecting the motors directly to a ring gear on the two 11 ft. 6-in. sheaves over which the cables pass. One end of each cable is attached to the span and the other to the counterweights. The friction between the cables and the sheaves causes the span to raise and lower when the sheaves rotate. There is no slip in the cables.

The four 40-h.p. motors (two on each tower) are synchronized in pairs. As only one pair of motors is required to operate the lift bridge, the brakes on the other pair act as emergency brakes. One pair of motors is used to operate the bridge on alternate days so as to insure the correct performance of either pair at all times.

The operation of the span is controlled from the operator's house which is on a level with the roadway, through a single drum-type controller. The controller is so connected that between the position of "nearly open" and "open", "nearly closed" and "closed", which represents about 5 ft. of vertical movement, only a small amount of current can be furnished to the motors. This prevents the sudden seating of the bridge by a careless operation, which would be serious because the weight of the span and counterweights is about 2 000 000 lb. The span is also equipped with air buffers at top and bottom.

The lift span is equipped with a self-leveling device operated by motor-generator sets such that, when the bridge becomes 18 in. out of level, additional resistance is automatically thrown in series with the motor, thus slowing down the motor on the side that is leading.

In addition, in case the automatic device breaks, there is an indicating dial, 18 in. in diameter, placed at the level of a man's eyes over the control drum. This is equipped with two superimposed hands, one red, indicating the position of the far end of the span, and the other black, indicating the position of the near end. Just below the dial are two push-buttons, one painted red corresponding to the red hand, and the other black, so that the operator can throw in additional resistance in the circuit with the leading motor, thus slowing it down. It is necessary for the span to get 6 ft. out of level before the guide-shoes on it will jam. Traffic is protected by alarm bells, blinking lights, and safety gates about 200 ft. from the lift span.

R. C. WILSON, ESQ., AND HERBERT B. POPE, ASSOC. M. AM. SOC. C. E. (by letter).—The discussions have really amplified the contents of this paper and, therefore, there is no need for a reply by the authors except to express appreciation to those who submitted them.

Mr. Ballard has brought out very clearly the fact that the various formulas in general use cannot be relied upon to govern the driving or determine the safe carrying capacity of the large concrete piles which were used in connection with this work.

The discussion by Mr. Willoughby has clarified certain details of design that had merely been outlined in the paper. His comments in regard to the machinery for operating the lift span are of particular interest in view of the first applications of direct drive to large lift spans.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

TRANSACTIONS

Paper No. 1769

FACTORS GOVERNING THE LOCATION OF AIRPORTS[1]

BY DONALD M. BAKER,[2] M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. STEPHEN CHILD, MORRIS KNOWLES, GEORGE D. BURR, N. H. ANGELL, J. OGDEN MARSH, ARCHER R. NORCROSS, JOHN NOLEN, S. N. GRIMM, R. U. ST. JOHN, JOHN F. PARTRIDGE, WILLIAM J. FOX, PERRY A. FELLOWS, W. W. CROSBY, E. K. SMITH, CLYDE M. WOOD, AND DONALD M. BAKER.

SYNOPSIS

The rapid growth of aerial transportation in the United States since about 1924, and, particularly, the wide interest shown since the spectacular trans-atlantic flight of Colonel Lindbergh, have developed the need of landing fields and airports in many communities. It is the purpose of this paper to outline, briefly, those things which should be taken into consideration when selecting a site to be used as an airport. The factors are discussed in detail and a method of rating is suggested by which various sites under consideration may be compared, one with another.

Matters pertaining to layout or design of fields are not discussed, except as they may affect the general size. While the subject-matter herein presented applies primarily to airports which will be used by heavier-than-air land planes, much of it applies also to the selection of terminal facilities for seaplanes. The development of lighter-than-air craft, or dirigibles, has not yet progressed to the stage where it has become necessary to provide facilities for them upon a large scale, and sufficient experience has not yet been acquired in their operation from a commercial standpoint to warrant the consideration herein of landing fields for this type of aircraft.

---

[1] Presented at the meeting of the City Planning Division, San Diego, Calif., October 4, 1928.

[2] Cons. Engr., Los Angeles, Calif.

Air transportation offers the human race a new means of mass transportation, in addition to the railroad and the automobile. The civil engineer has played an important part in developing railroads, highways, and terminals, and he has his proper place to take in the field of air transportation. Travel by air is more closely akin to water transport than to land transport. The right of way is not rigidly fixed, and the only developments necessary in the establishment of airways are provisions to insure safety in flight and maintenance of proper direction, plus terminal facilities. Just as ships must be provided with harbors or docking facilities, so must aircraft be provided with landing facilities.

An airport, however, must be considered as more than a mere landing field for aircraft. It is a real terminal or harbor equipped with all the necessary facilities and utilities for handling aircraft, for passengers and cargo, for storage and repair of planes and engines, for testing them, etc. It must have all the characteristics and accessories of any terminal as well as other features peculiarly incident to air traffic.

The users of an airport may be roughly classified as:

1.—The Federal Government, for military and naval planes, etc.
2.—Commercial transport organizations.
3.—Operators of private, pleasure, and business planes.
4.—Flying schools.
5.—Operators of commercial and sightseeing planes.
6.—Aircraft manufacturers and allied industries.

At present, aviation occupies the status of an "infant industry" operated in small units. As such it merits public subsidy. The acquisition and operation of airports by public agencies, such as cities and counties, are fully justified on the same grounds as the public development of harbors and highways. Where nominal fees are charged for the use of space and facilities of an airport, its public development forms a most reasonable and logical type of local subsidy. When traffic increases in volume, as it is bound to do, there is little question but that publicly owned airports will become self-sustaining.

USES OF AIRPORTS

The primary excuse for the acquisition of an airport by a public body lies in the need for assisting and encouraging air transportation. The question will naturally arise as to whether one or more airports should be acquired in any community. Various estimates have been made concerning the future density of aircraft in this country, ranging from 1 ship per 100 persons to 1 per 1 000.

In the smaller communities undoubtedly all types of aeronautical activities will be concentrated in one airport. However, in larger centers, if not now, at least in the near future, it will be essential to segregate the different types of use, selecting one large site to serve as a major traffic terminal, testing and industrial field, and, possibly, a passenger and cargo depot, with one or more additional sites for private, commercial, and sightseeing planes, flying schools, and similar uses. Where this possibility exists, the subject should be given considerable study by the local authorities, and a program adopted for the establishment of an adequate number of airports to serve the area.

SELECTION OF SITE

To the average layman or amateur any large open area, fairly level, ranging in size from 100 to 1 000 acres, near the city, and available at a price, is a suitable site for an airport. On the contrary, the selection is a highly technical problem involving knowledge of aviation, city planning, transportation, meteorology, finance, and law, and should only be attempted after adequate investigation and study by experts thoroughly familiar with the subject.

Inasmuch as the capital outlay for land to be used for an airport site is considerable, and usually forms a large proportion of the total investment, and as a proper location for such a site will have much to do with the successful development of aeronautics in the community, the small cost and even short delay incident to technical study and investigation prior to purchase always will be warranted.

Where meteorological conditions vary greatly within a radius of from ten to twenty miles from the center of the community, it is desirable that a study of such conditions be carried on at all sites under investigation for a considerable period—if possible, a year. This study should be directed toward securing data on wind direction and velocity, and variations of these factors, visibility, occurrence of fog, rainfall, air turbulency, and related matters. If time is short a study of these elements for a period extending over a few months will be of great value, particularly when the results can be correlated with data from the U. S. Weather Bureau Stations in the vicinity. Other factors, such as present and future accessibility, transportation, trend of surrounding development, etc., do not take as long to investigate, and their study may be carried on while the meteorological observations are being made.

FACTORS GOVERNING LOCATION OF SITE

Factors affecting the site group themselves into three classes, namely, those dealing with the general location, those dealing with the site itself, and those dealing with the primary use to which the site is to be put.

After it has been determined as to whether one or a number of airports are to be acquired, and also as to the respective use or uses which will be made of each location, the sites should be studied individually from the following standpoints:

(*a*)—Topography.
(*b*)—Physical Features.
(*c*)—Meteorology.
(*d*)—Location with Respect to Community Served.
(*e*)—Transportation Facilities.
(*f*)—Accessibility.
(*g*)—Legislative Control.
(*h*)—Utilities Available at or near the Site.
(*i*)—Cost.
(*j*)—Miscellaneous.

These factors will be discussed in detail. The requirements set up are basic for any site, except where noted, and are the maximum to be desired.

(*a*)—*Topography.*—Topography includes both that of the site and that of the surrounding territory.

The surface of the landing field should be flat, or of such a character that it can be made flat at nominal cost. Rolling or broken country is undesirable on account of grading costs. The surface of the field should have a general slope to facilitate drainage, ranging from a minimum of 0.5% to a maximum of from 1.5 to 2.0 per cent.

Since most accidents to planes occur within the first few hundred feet after they leave the ground, open spaces to the windward of the airport are desirable for use as emergency landing fields in case of engine trouble or other difficulties occurring immediately after the take-off.

Distinctive landmarks should exist, such as stream channels, highways, railroads, and similar topographical features. These will readily identify the airport to the strange flier approaching the field from the air, and will also assist the familiar pilot in locating it in bad weather.

The surrounding topography should be of such a nature that reasonable future expansion of the airport can be made at a cost which will not be prohibitive.

(*b*)—*Physical Features.*—The site must be protected from overflow by flood waters. The surface of the landing field should be kept free from mud or standing water at all times; this requires a quick draining soil of light texture underlaid by an open subsoil. Clay or other heavy soils that do not yield moisture readily may require surfacing or an expensive tile drainage system.

Where a field is used extensively it will become dusty in dry weather if the soil is of a friable character. Preventive measures in this respect are quite costly. In uncultivated country, wind storms may cause objectionable dust haze with resulting poor visibility.

If turf or sod are to be used on the runways or landing areas, a fertile soil is necessary and, where rain does not fall throughout the year, water must be available for irrigation. With other soil, landing strips must be built of concrete, cinders, slag, gravel, asphalt, or other suitable material.

High-tension wires, buildings, poles, and smokestacks in the vicinity of the site constitute serious hazards and are very expensive to remove.

Care must be given to the location of future highways, as no traffic arteries should be allowed to cross the site, and if the airport is located in the path of such an artery, expensive location around the site may become necessary.

The U. S. Department of Commerce has developed a system of rating airports, based on the following:

1.—General facilities and equipment.
2.—Size of landing area.
3.—Night lighting equipment.

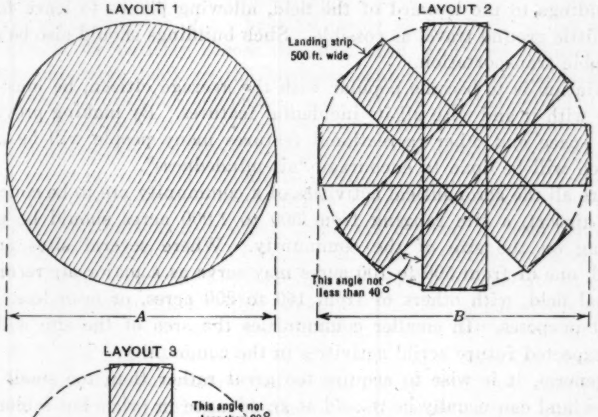
Six ratings for size are given as will be shown. In addition, the following basic requirements must be met:

(*a*) The field must be approximately level, with a maximum slope at any one point of less than 2.5% and an average slope not in excess of 2 per cent.

(*b*) Clear approaches must exist from all sides of the field so that no obstacle projects above a 1 to 7 grade from the exterior boundaries of the field.

Certain ratings have been adopted by the Aeronautics Branch of the U. S. Department of Commerce, as to the site of landing fields.

For Rating 1 (Fig. 1), the field must have 2 500 ft. of landing area in all directions; or landing strips, 500 ft. wide, permitting landing in at least eight directions, the strips to be at least 2 500 ft. in length and crossing and converging at angles of not less than 40 degrees.


LAYOUT 1
LAYOUT 2
Landing strip 500 ft. wide
This angle not less than 40°
A
B
LAYOUT 3
This angle not less than 60°
Landing strip 500 ft. wide
C

Rating	Dimensions
1	*A* and *B* = 2 500 ft.
2	*A* and *B* = 2 000 ft.
	or *C* = 2 500 ft.
3	*A* and *B* = 1 600 ft.
	or *C* = 2 000 ft.
4	*A* and *B* = 1 320 ft.
	or *C* = 1 600 ft.

FIG. 1.—DIMENSIONS OF LANDING FIELDS FOR RATINGS BY THE U. S. DEPARTMENT OF COMMERCE.

For Rating 2, the field must have 2 000 ft. of landing area in all directions; or landing strips, 500 ft. wide, permitting landing in at least eight directions, the strips to be at least 2 000 ft. long and crossing or converging at angles of not less than 40°; or two landing strips, 500 ft. wide and 3 000 ft. long, permitting landing in four directions, the strips crossing or converging at angles of not less than 60 degrees. Comparable details for Ratings 3 and 4 are also given in Fig. 1.

Rating 5 includes fields not having minimum dimensions of a No. 4 rating for size, but which the Department considers safe for the use to which they are put.

Finally, fields not considered safe for operation of aircraft, but desiring a rating, will be rated "0" on size.

The dimensions given must be increased where fields are at altitudes in excess of 1 000 ft. above sea level.

In addition to the area necessary for runways, space is required for shops, hangars, and administration buildings, and if the airport is to be used for industrial purposes it may be advisable to provide space which may be leased to these industries. It is advisable to locate shops, hangars, and administration buildings to the leeward of the field, allowing planes to leave for flying with as little ground travel as possible. Such buildings should also be adjacent to available transportation.

If aviation is to become popular with the average citizen, he must become familiar with it and with all its incidental features. By making provision for parking space and other recreational features many people will be attracted and much will be done to encourage "air-mindedness".

Where all the aeronautical activities of a community are to be concentrated on one airport, a site ranging from 500 to 1 000 acres should be provided, depending on the size of the community. Where several sites are to be acquired, one of from 500 to 700 acres may serve as a major air terminal and industrial field, with others of from 150 to 200 acres, or even less, used for different purposes. In smaller communities the area of the site will depend on the expected future aerial activities in the community.

In general, it is wise to acquire too great rather than too small an area, as excess land can usually be re-sold at an advance in price, but seldom can be purchased after the airport is developed, except at prohibitive prices.

(*c*)—*Meteorology.*—In localities where little variation in meteorological conditions occur within a radius of from ten to twenty miles, these conditions must be taken as found, even if unfavorable; but where much local variation exists, meteorology is an important factor.

Haze in the air caused by wind, dust storms, or smoke is very objectionable, preventing pilots from locating the field, contributing to loss of time in landing, and possibly causing accidents. Sites to the leeward of manufacturing districts, particularly where coal is used as a fuel, or those in large areas of uncultivated or sandy soils, will have poor visibility.

Fogs are of great menace to aerial navigation. Ground fogs hinder safe landing and take-off, while high fogs cause loss of bearing by pilots, and delay in arrival. Airports located in fog belts are extremely undersirable for use by commercial or transport planes which operate on a fixed time schedule, although they may not be quite so objectionable to occasional fliers, sightseers, or pleasure planes, which can choose their time of flight.

A plane must take off and land, flying directly into the wind. Sites which are subject to quickly shifting winds require great care in keeping all the runways clear. Strong winds may cause damage to planes parked outside of hangars and necessitate increase in storage facilities. "Bumpy" air, caused by cross-air currents which result from unevenness of surface topography or nearby hills, makes for difficulty in control of ships, and is extremely dangerous where the field is congested.

(*d*)—*Location with Respect to Community Served.*—The airport should be located so that flying over the residential section of the community can be kept at a minimum. There is an increasing tendency in cities to prohibit air traffic over residential areas except at very high elevations, or to restrict air travel over certain routes. These restrictions will impair the efficiency of any field within or near such residential areas, or limit the line of expansion of such development.

By studying the major directions of approach of future air traffic, the site may be located in the line of the heaviest traffic, as time will be lost if planes are required to fly entirely across the city, land, and then transport their passengers and cargo back to the business center. Smaller fields used for private planes, sightseeing companies, etc., can be located adjacent to the better class residential area so as to afford ready access for passengers.

(*e*)—*Transportation Facilities.*—If possible, the airport, particularly one used for a major air terminal, should be located adjacent to rapid transit lines and major highways leading directly to the community served. It is improbable that sufficient passenger or other traffic will develop, at least for years to come, to warrant the extension of existing rail lines to connect with an airport. The usual schedule maintained by a steam road is seldom adequate to serve the field, but rapid transit lines, either steam or electric, generally maintain a schedule which in frequency of trains and running time will satisfy all requirements. With the future increase in air traffic and resulting decrease in carrying charges for passengers and cargo, cheap and fast transportation between the airport and ultimate destination will be demanded, and this only rapid transit can furnish. If such cannot be made available, a regular motor transport line will ultimately have to be provided.

While freight facilities are desirable they are not absolutely essential, provided the site is served by highways. Practically all the cargo carried will be of the express class. Heavy material and supplies used in the construction and operation of the port can be transported by motor truck. Motor bus lines supplementing rapid transit facilities, if available, are a feature of added value.

(*f*)—*Accessibility.*—Aerial transportation will be dependent largely for its success on the element of time saving over rail or motor transport. Present schedules now average about 100 miles per hour, and these will shortly be increased by 25%, or more. The time element must be considered, however, as being from point of origin of trip, by passenger or cargo, to the point of ultimate destination. This time divides itself into that consumed between the airport and points of origin and destination, and that consumed by air travel.

In time, any site served solely by highway transportation will ultimately become farther and farther away from the business center, due to increasing traffic congestion, as American cities do not appear to be able to provide highway facilities fast enough to keep ahead of the increasing density of motor traffic. A site served by rapid transit, however, will always remain at a constant time distance from the business center, and with improvement in rapid transit facilities may approach closer to it in point of time.

For short distances between ultimate termini, the time saved by air over rail transport is small. The curves of Fig. 2 indicate that great saving in time through use of air over rail transport only occurs on long hauls, probably in excess of 500 miles for passenger travel and 1 000 miles for other cargo. The added cost of air over rail transport will undoubtedly limit its extensive use to the longer hauls.

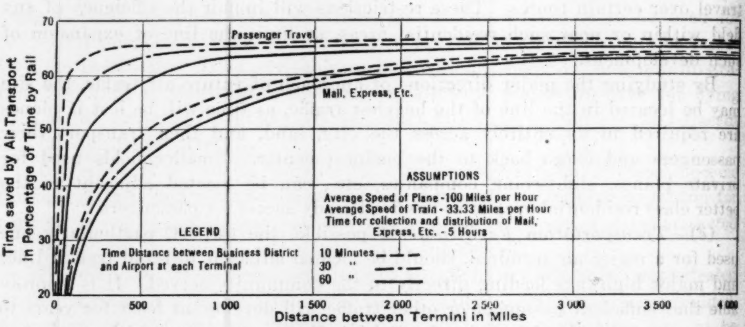


FIG. 2.—SAVING IN TIME ACCOMPLISHED THROUGH USE OF AIR TRANSPORT INSTEAD OF RAIL TRANSPORT.

These curves also show that undue weight is given to the necessity of locating an airport which is to be used as a major traffic terminal close to the business center. Land values in general decrease with an increase in distance from the center of the community. A careful study must be made of the relationship between the time saved through the selection of a close-in site as against the increased cost of such location. This point is of great importance.

Due to the psychology of the air traveler, however, it is not advisable to locate an airport more than from 30 to 40 min. from the business center if it is to be used as a major air terminal, unless land values at this distance are prohibitive, in which case a small close-in port may be chosen as a depot with a larger field for service, storage, etc., selected in an area of cheaper land.

Sites used for private planes should be within fairly close distance of the residential communities. Otherwise, the extensive use of planes by private parties will be discouraged. The plane owner does not care to spend an hour traveling out to the airport for a flight of like duration.

(g)—Legislative Control.—Police protection and regulations should be available at the airport. The development of the area surrounding the site must be subject to such control as will prevent the encroachment of residential areas or of obnoxious industries which might impair the usefulness of the field. It is not essential that the airport be within the corporate limits of a city which owns and operates it, provided proper authority exists allowing

ownership and operation of such facilities outside of city boundaries, and that proper control may be had or secured over the policing, zoning, building regulations, etc., in the area surrounding the site.

(*h*)—*Utilities.*—Any site considered should have electric power, water, fuel, telephone, and telegraphic connections with surrounding centers, and should be accessible to supplies of gasoline, oil, and other necessities. Where the field is to be used for industrial purposes, or when workmen are to be employed, housing conditions for workers may be an important consideration.

(*i*)—*Cost.*—To afford a proper basis for comparison between different sites, and to evaluate their relative accessibility, there should be included in the cost of each site, in order to reduce such cost to a common denominator, the following:

1.—Land.
2.—Grading, leveling, surfacing, draining, and tiling.
3.—Removing hazards and obstructions.
4.—Bringing transportation and traffic facilities adjacent to site.
5.—Bringing necessary utilities adjacent to site.
6.—Providing adequate fire protection.
7.—Financing, condemnation, etc.

(*j*)—*Miscellaneous.*—Undoubtedly, certain other considerations entering into the problem of selecting a site from a local standpoint, may be peculiar to the community to be served, and do not come under the headings given. Local politics, while they should be kept out of consideration, nevertheless are a factor in it. All these latter matters are grouped under the heading of miscellaneous factors.

Basis of Comparison of Different Sites

It will be well-nigh impossible to find a location for a municipal airport which is perfect from all standpoints. When various sites are under discussion, certain factors are likely to be given undue prominence in reaching a decision, and the following method is suggested as a means of arriving at the most desirable site.

Every site should be considered separately from each of these ten standpoints, and each factor given a rating on the following basis:

Rating.	Percentage.
Excellent	90 to 100
Good	80 to 90
Fair	70 to 80
Unsatisfactory	Below 70

Little consideration should be given a site where factors of meteorology and accessibility fall below 90%, or where topography, physical, or transportation features fall below 80 per cent.

Since not all the factors considered are of equal importance they should be given different weights. The following are suggested as suitable of application where a single airport is to be selected and used for a major traffic terminal:

Factor.	Weight.
(*a*).—Topography	10
(*b*).—Physical features	14
(*c*).—Meteorology	14
(*d*).—Location with respect to community served	10
(*e*).—Transportation facilities	11
(*f*).—Accessibility	11
(*g*).—Legislative control	5
(*h*).—Utilities available at or near site	5
(*i*).—Cost	10
(*j*).—Miscellaneous	10
Total	100

By multiplying the percentage rating given each factor for the site under consideration into the weight assigned that factor, and taking the total of the resulting figures, a final rating for each site can be found. The weights given may be varied somewhat, depending on the use to which the site is to be put.

DISCUSSION

STEPHEN CHILD,[3] **M. AM. SOC. C. E.**—Tucson is a thriving city of about 40 000 population, in Southern Arizona, approximately seventy miles from the Mexican line. It has been actively and definitely "air-minded" for some time, as the following facts furnished the speaker by the Secretary of the Tucson Chamber of Commerce, will abundantly testify.

The first municipal aviation field in Arizona, if not in the United States, was established at Tucson, and began to operate as such in 1917. It had an area of 80 acres and cost $7 000. It is about 2 miles directly south of the city on a site selected by Eric Nelson, one of the world-famous fliers. About 1922, the Chamber of Commerce, in co-operation with the City, erected a hangar and a building for quarters for the employees who were furnished by the U. S. Army Air Service, and a lease was made by the City to the Air Service for the operation of the field.

This original airport of Tucson is still owned by the City and is leased to a commercial aviation company, which is offering to the public aviation training as well as taxi service. This provision meets the requirements of Mr. Baker, who recommends that such features should not be permitted at the more official airport of the city.

In 1925, however, the Chamber of Commerce took much more important action and was able to secure the passage of a bill by Congress, setting aside two full townships (1 280 acres) only a little more than 4 miles from the center of the city. This land has been leased to the City of Tucson for twenty years, with an option to renew the lease for another twenty years, for the exclusive purpose of developing aviation. It will be seen that the important provision of permanency stressed by the author, is assured.

The tract is very nearly level, having a gentle downward slope from east to west of only about 1 per cent. There is no obstructing vegetation, no ledge or boulders to be removed, or important gullies to be filled up. There is merely the customary desert growth of mesquite and cactus. The topographical conditions, therefore, are nearly perfect.

Of this large area 240 acres have now been quite fully developed by the City of Tucson and the Army Air Service, at a total cost of about $30 000—a minimum figure. A hangar has been erected, attendants' quarters have been constructed, a pumping plant for water supply has been installed, and a Government Radio Meteorological Station has been set up and fully equipped. By means of a public subscription, aided by the City Government, sufficient funds to install a beacon light and flood lighting equipment have been raised. The Army Air Service keeps a sergeant and one private stationed at the port to service all Army planes. A large gas storage equipment has been installed by the Government, and private oil companies have set up storage tanks on the field for commercial planes. The equipment is, therefore, practically perfect for a port of this size, and there is ample opportunity for its extension.

---

[3] Landscape Archt.; Consultant in City Planning, San Francisco, Calif.

The airport is used jointly by Government and commercial planes, under the direction of the Army Air Service and the supervision of the City itself. Legislative control as emphasized by Mr. Baker, therefore, is well provided.

As to accessibility it will be noted that the Tucson Airport is only about five miles from the heart of the city over existing rectangularly arranged roads, which are well paved. It should be emphasized, however, that the main line of the Southern Pacific Railroad crosses this airport at its southwest corner, and that there is now a flag station stop at this point, which can be readily made more important when the need arises. There is no doubt that there will be constructed in the near future a diagonal avenue parallel to the existing railroad tracks, shortening the distance from town by at least a mile.

One of the well-known aviation transportation companies operates a passenger service between this port and Los Angeles, Calif., three times a week, a stop being made at Phoenix, Ariz., en route. Connections have been established so that it is possible now to leave Tucson at 8:00 A. M., arriving at Los Angeles at about 1:00 P. M., and connecting with the airplane service operating between Los Angeles and San Francisco so as to arrive there in the early evening.

Leases for hangar space are arranged with the City and the airport is now so well known and completely equipped that it has a double "A" rating, and is considered one of the best inland airports of the country. Between January 1 and October 1, 1928, the registration of the airport shows more than 900 airplanes using its facilities, and during the month of September, 1928, 275 were registered. The City of Tucson has appropriated for the fiscal year 1928-29, funds for the construction of a building to be used as a waiting-room, ticket office, and lunch room.

The author has suggested a means of comparing different sites, and the speaker has thought it interesting to develop the rules given, as shown in Table 1.

TABLE 1.—COMPARATIVE RATING OF AIRPORT AT TUCSON, ARIZ.

Factor.	Weight.	Percentage.	Rating.
(*a*) Topography	10	100	1 000
(*b*) Physical features	14	100	1 400
(*c*) Meteorology *	14	95	1 330
(*d*) Location with respect to community served	10	95	950
(*e*) Transportation facilities	11	95	1 045
(*f*) Accessibility	11	95	1 045
(*g*) Legislative control	5	100	500
(*h*) Utilities available at site	5	100	500
(*i*) Cost	10	90	900
(*i*) General desirability from all standpoints	10	95	950
Total	100	....	9 620

* No fogs; sun shines on an average of 358 days in the year.

In other words, the field is 96.2% ideal, which is a very high rating. It will be interesting to determine the ratings of other airports by this means of comparison. The speaker hopes that Mr. Baker will submit such comparisons, but it seems clear that for a comparatively small community of less than 40 000, the facts indicate a very creditable state of affairs for Tucson as to aviation and airports.

MORRIS KNOWLES,[4] M. AM. SOC. C. E.—It is well to emphasize the fact that airport location, like other new municipal problems, is simply one of city planning, and as such it should be solved. It is inconceivable to consider an airport as wisely located, if in the selection of a site the fundamental principles of good community planning have been violated. This has been stressed by Mr. Baker. A definition has been given[5] that:

> "An airport is a locality, either water or land, adapted for the landing or taking off of aircraft, and which provides facilities for shelter, supplies, and repair of aircraft; or a place used regularly for the receiving or discharging of passengers or cargo by air."

Safety and convenience, in the provisions for the landing or taking off of planes, are matters of first importance. However, aside from possible emergency fields, which may be for this purpose only, an airport assumes an objective for traveling planes other than that of landing; or to reverse the situation, a plane in the ordinary course of events must have some object in being at a certain location other than the mere fact of desiring to leave there. In short, this reduces to the statement, by way of emphasis, that while air transportation may be considered more or less in its infancy, its real importance as a public carrier certainly will increase and, with it, the necessity of locating the airport with respect to traffic-handling facilities and other similar features.

Mr. Baker has stressed ten features requiring consideration in determining ratings for locations offered as sites for airports. In reference to five of these, he states that little consideration should be given to a site which does not measure up to 80 or 90% of the standards given for them in the tables. Although he does not say so, it may be assumed that he considers others of less importance. This suggests the possible elimination of such minor items as Factors (*g*) and (*j*) in the rating and the substitution of others of more importance. For example, there are two such items discussed under the heading of "Topography" (Factor (*a*)): Those of open spaces for emergency landing fields and landmarks for identification.

The selection of a site for the Pittsburgh (Pa.) Airport, by a Committee of the Chamber of Commerce, was made by the use of a rating schedule, developed after approximately two hundred sites had been listed for consideration. Many of these had been suggested by the public through a questionnaire which was sent out for the purpose. This, in order to confine suggestions within reasonable limits, was made up so as to attract attention to the salient factors which would be considered in the choice.

The study of this large list of possibilities was simplified by the preparation of charts which enumerated, on a comparative basis, the desirable and undesirable features of each site. This naturally eliminated many because of some outstanding objection or obstacle. The rating schedule was then developed and applied to the more desirable sites after they had been inspected from the ground and air. Thus, as the study progressed, eliminations increased, until only a few sites remained for consideration.

---

[4] Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

[5] *Bulletin No. 40*, U. S. Dept. of Commerce.

Such a rating table suggested as an aid in the final analysis, gives the weights of the various factors considered as important under the particular circumstances. It is as follows:

		Weight.
(1)	Size, physical features, and freedom from obstructions..	20
(2)	Distance from business district and City Post-Office....	15
(3)	Accessibility by various transportation methods........	14
(4)	Possible emergency fields existing near-by............	12
(5)	Railroad connections, with sites for shops............	10
(6)	Freedom from wind, fog, and smoke....................	9
(7)	Property and cost of converting (quite approximate)...	8
(8)	Seaplane facilities in proximity........................	5
(9)	Distinguishing landmarks for visiting pilots..........	4
(10)	Water supply and fire-fighting facilities..............	3
	Total	100

While such ratings form an important basis for selection, many general factors will still remain to be evaluated, and the final recommendation should depend on the experience and judgment of those who are attempting to make the selection.

In discussing the development of airports it is important to realize that the problems differ in coast and inland cities, not only because of topography, but because of location. For example, Mr. Baker emphasizes the desirability of locating the field on the side of the city from which the aircraft will approach. This is not important for most inland locations, as the approach may be from any direction and, probably, in the majority of instances, unless detours are made, it will be necessary to fly over the city either approaching or leaving the field. At the same time it is possible that where the routes are at right angles—east and west and north and south—a location adjacent to the city at one of the intermediate points of the compass would make it possible to avoid flying over the built-up territory.

Mr. Baker refers to the undue weight which often is given to the necessity of locating an airport close to the business center. This undoubtedly is true. However, not only is the element of time important, but the cost of additional transportation as well. Studies in some places have emphasized the fact that the cost of the additional hauling for mail, express, and passengers, over a distance of 10 miles as compared to 2 miles, may approach the amount of interest on the additional money which would be needed to purchase and develop a field near the center of the city. Commercial aviation perhaps has not yet reached the point, in this country at least, which would economically justify the clearing of a built-up area in the city for an airport. Nevertheless, such conditions may be anticipated with the perfection of helicopters, just as it has been found necessary in the past in the rebuilding or relocation of railroad terminals.

The City of Pittsburgh has some unusual problems in the development of an airport—because of topography and the difficulties of making a field relatively level—but when these have been solved it will have some peculiar advantages because the field will be high in the air and will overlook most of the surrounding country. This elevation is important in regard to the question of fog and smoke; it also makes it possible to provide sufficient graded land, adjacent to the field, so that the parking of automobiles and the erection of shops and convenience buildings can take place without infringement on the clearance adjacent to the field. In other words, because of the resultant lower elevation for such purposes, there is no necessity for removing structures, or for parking cars, seven times their height horizontally from the field; or, as an alternative, decreasing the effective area of the available space by this amount.

Referring to the requirements of ample land area for future growth, it is notable that no mention is made of that for the parking of automobiles. This is an important necessity and there should be ample provision on four sides of the field, so that at times of great public demonstration cars may enter and leave the parking area without interference with other traffic. It is well to emphasize this need of provision for ample area, at least for later expansion, if required. The probabilities are in favor of additional land being needed for industries associated with air transportation, and for similar purposes. Aside from every other consideration, there is one convincing thought which gives assurance as to the probability of providing too large an area in view of anticipated improvements in airplane design. In the history of cities in the United States there have been few, if any, instances where too large an area has been set aside for public use.

One more item to which it seems desirable to call attention is that of the provisions for seaplanes. While this is an important matter for sea-coast towns, and those along the Great Lakes, it does not give the impression of being important for inland cities, even though they may have a good river harbor. The infrequency of such travel (due to the fact that most inland cities must utilize other forms of airships and that the interference of bridges is likely to be troublesome) does not suggest that much weight need be given to this element.

George D. Burr,[6] **Assoc. M. Am. Soc. C. E.**—The subject of the proper location of airports is a timely one. It is perhaps too early to attempt to assign definite values or weights to the factors that must be taken into consideration in the intelligent selection of an airport site.

There is only one thing that commercial air transportation has to sell and that is speed, or time saving. The one factor on which the sale of this commodity hinges is safety and certainty of arrival on time. Therefore, a commercial or municipal terminal airport must be located as close to the metropolitan center of the city as safety will permit.

This rule is quite simply stated and quite difficult to apply. However, with existing commercial types of heavier-than-air craft, no site will be satisfactory regardless of other conditions: (1) If meteorological conditions are unsafe; (2) if the site is too small for safe landings or take-offs; (3) if dangerous flying practices are permitted over, on, or near the field; or (4) if the field is a long-time distance from the ultimate termination of the journey of passengers or commodities.

---

[6] Associate Engr., U. S. Dept. of the Interior, National Park Service, San Francisco, Calif.

The cost of purchasing land, bringing utilities to the site, developing transportation facilities, improving difficult drainage, surfacing runways, leveling land, and similar factors, might be grouped under one single head, namely, cost of site.

If the nearest safe site that may be developed economically is a relatively great distance from the ultimate destination of passengers traveling to the city served it might be used, but always with the thought in mind that, although it is an aid to air navigation, another site, closer to the final destination, will some day be demanded when air transportation has developed sufficiently to make such more expensive sites economically feasible. However, if a site is close in, but is decidedly unsafe, because of meteorological conditions or small area, that site will be abandoned in the future.

These considerations would leave only three items to be considered in the selection of a terminal airport as follows:

Factor.	Weight.
(*a*) Safety, with consideration of fog, smoke, wind, turbulence of air, and safe area of field	Mandatory.
(*b*) Distance from ultimate destination	As close as feasible in consideration of safety.
(*c*) Cost	Select the site that is cheapest to develop if two or more sites present the same relative advantages from the point of view of Factors (*a*) and (*b*).

It must be borne in mind, however, that these remarks appertain to the selection of a terminal commercial or municipal airport for heavier-than-air craft only. There are many more classes of airplane landing fields to which these principles will not apply. For instance, there are: (1) Emergency and intermediate fields placed along an airway; (2) fields developed for the sole purpose of providing a training place for students; (3) fields located primarily as a junction and transfer point for air mail lines with little local delivery; (4) fields used principally for some specific commercial purpose, such as for tests by an airplane manufacturer, or as a terminal point for an air mapping company, or an air "taxi" or "joy hop" recreation center; and (5) fields devoted almost exclusively to the storage of privately owned craft used for private business or pleasure. The present tendency is to specialize in the use to which a field is to be put.

The question of just what amount of money may be justifiably expended on a given site by a given community or commercial enterprise is a matter for definite analysis. Eventually, the airport must pay a return on the capital invested.

There are several sources of revenue from a publicly owned airport enterprise. These are rental of hangar space; rental of concessions, such as restaurants, gas stations, etc.; rental of office space; charges for use of special facilities; and charges for landings. At present, charges for landings are seldom imposed, in order that flying may be encouraged. Hangar rentals and office rentals form the principal, immediately available, source of revenue.

A charge of 3½ cents per sq. ft. per month, and sometimes more, for hangar space is now quite generally imposed. Hangar space may be constructed for approximately $2.50 per sq. ft., including all costs, exclusive of land. If such hangars bring a return of 2½ cents per sq. ft. per month, an adequate return is made to amortize the investment in the structure within a reasonable time, and to pay interest, repair, and depreciation charges. The remaining 1 cent per sq. ft. per month of return may go to help pay field operating expenses, maintenance, and purchase cost of land. This specific feature has been elaborated somewhat in order to show the nature of the financial problem.

N. H. Angell,[7] Esq.—In this paper a rating of 14 points was given to the physical features of the airport. The factors which would influence the percentage of rating under this classification would evidently be the condition of the surface of the field, and the manner in which the field is laid out. The latter has been well covered in *Bulletins* of the U. S. Department of Commerce and by the author.

The question of surfacing has usually been left to the individual fields to work out for themselves. It is a question of special interest on the Pacific Coast. There is no doubt that some kind of a surfacing is necessary. Due to the lack of rainfall in summer months and the consequent cost of irrigation, turf fields have been virtually prohibited in the Far West. The development of some other type has been necessary.

Aside from the question of elimination of dust, the functions most desired in runway treatment are that it shall provide a firm surface over which planes can accelerate to the desired flying speed in the shortest distance, and that this be done with comfort to passengers at all times when weather permits flying. With the flying speed of planes ranging between 50 and 75 miles per hour, the take-off can be made in a shorter distance when the run is over a firm, even surface than when it is attempted from a rough, irregular, and perhaps muddy field.

Increased passenger carrying is demanding surfaced fields. In the larger airports constructed by aviation corporations which have sufficient money available, the surfacing of the field is one of the major items considered, and in many cases the field is not opened until such surfacing has been completed.

The use of hard surfaces on landing areas is not regarded with favor by most pilots. A comparatively soft, resilient surface is preferred, one that will have an appreciable tractive resistance and aid in slowing down the plane, as well as help to absorb the shock of impact. The drag of a tail-skid on a surface of the proper consistency aids materially in decreasing the run after landing, although its primary function after the plane is on the ground is to exert a stabilizing effect and eliminate any tendency of the plane to veer from one side to the other or "ground loop." In order to insure this desired stabilizing effect, a somewhat soft or plastic landing surface is preferred. Properly constructed, this surface is also suitable for take-offs.

---

[7] San Francisco, Calif.

Asphaltic oils with a properly graded sand have been used for the construction of such surfaces. The treatment may consist of a surface application in which the oil is allowed to penetrate, or if more money is available, analysis of the soil may be made and, with the addition of foreign sand to supply any deficiency in the grading of the natural soil of the airport, an oiled sand mat may be built up by successive applications of oil, each application being mixed with the sand by harrowing and blading. This is similar to the "mixed-in-place" method used in the construction of secondary highways in many of the States, except that the grading of the aggregate is quite different. When practical, it is preferable to mix the surface in an asphalt plant in much the same manner as the ordinary asphaltic concrete or sheet asphalt types of surfacing are mixed. This provides for an accurate control of the aggregate and oil at all times, insuring uniformity of the surface.

On the United Airport at Burbank, near Los Angeles, Calif., there are approximately 70 acres of runway surface treated with a pre-mixed oiled sand material. For this work the aggregate was in some cases as large as ½ in., grading down to 200-mesh material. There are three runways on this airport, each 300 ft. wide and averaging 3 500 ft. in length. The plant-mixed asphaltic surface is soft enough to be marked and even creased by tail-skids, but firm enough for take-offs under all weather conditions encountered. In addition, there are about 50 000 sq. yd. of asphaltic concrete, 4 in. and 3 in. in thickness, on roadways, warming-up platforms, and parking areas.

There are a great many airports in California that have oil-treated surfaces and the dust nuisance has thus been largely eliminated. Such treatment has resulted in increased revenue from passenger carrying and decreased maintenance costs on the airport surfaces and on equipment operated from the field.

Many fields have started in their surfacing program by paving the areas adjacent to the hangars, such as the aprons and warming-up platforms. The pavement has been made as thin as possible, consistent with the subgrade and the load to which it will be subjected. By using a minimum thickness, larger paved areas are possible from the same funds. It is felt that if there is an increase in the size of planes to such an extent that the pavement will not be strong enough, bituminous types can be easily thickened by a resurfacing course, without sacrificing the initial investment.

J. OGDEN MARSH,[8] M. AM. SOC. C. E.—When automobiles first began to appear, most people regarded them as a "rich man's toy", and not until the public took them seriously and as a commonplace part of daily life did the automobile industry begin to grow. The present state of the public mind in regard to aeronautics is somewhat similar, and in the speaker's opinion, there is nothing more important than to have the public become so familiar with airplanes that they will look upon them as ordinary functions of daily life.

---

[8] Chf. Engr. and Gen. Mgr., Board of Public Utilities and Transportation, Los Angeles, Calif.

The author has stressed the necessity of public support for the acquisition and construction of principal airports. The public does not expect steamship companies to pay for harbor development, and they probably should not expect air transportation to provide the airports. If aviation interests are to have public support financially, they must make the public "air-minded".

As airplane activities must naturally be centered about airports, the problem resolves itself into one of getting the public to take more interest in visiting airports, both public and private, and in actually seeing the operation of flying. To accomplish this, planning engineers must make airports not only extremely accessible, but they must make them attractive. There must be correlated with airports, and their surroundings, other and more familiar lines of interest and amusement that will bring people to these localities. This is a matter which should be given some thought, because it is important to get the crowds there. To do this with convenience, the necessary roads must be built for motor vehicles.

The desire, however, to attract crowds should prevent the absolute prohibition of stunt flying, as such flying fosters the public belief that airplanes are not a safe and dependable means of transportation.

ARCHER R. NORCROSS,[9] ASSOC. M. AM. SOC. C. E.—The author mentions the layout and construction of runways and gives several types of surfaces which have been advocated for runway construction, or have been already used on some airports. The proper surfacing of the airport is a problem which is being given constantly increasing attention and thought on the part of airport designers and constructors. While the surfacing must provide for an entirely new type of traffic, the methods used by highway engineers, and the lessons which they have learned in the development of durable and economical surfaces, may be well applied to airport work.

Runway surfaces must be firm enough to resist displacement under loads which are already up to 25 tons and may become greater. They must be free from any tendency to rut or break. In addition to the static load which they must stand, the force of impact must be considered. It would thus appear that the surfacing material should be of a type which is easily thickened to take care of the increased loading that may be expected as a result of the tendency for planes to develop along a larger scale.

The methods of the highway engineer as exemplified on the new construction work on secondary roads in the West may well be adapted to preliminary construction on runways. The application of an asphaltic oil with surfacing of fine screenings and the whole compacted by rolling so that a water-proof and firm surface is obtained, or some similar method of treatment, will not only provide a runway surface that will serve for some time, but it assures an excellent sub-grade for future paving.

---

[9] Brookline, Mass.

There is one item which enters into the construction of airport surfaces with much more importance than is attached to it in highway work, that is, the visibility of the surface. Oiled or dark bituminous surfaces appear to be regarded with most favor by pilots. It might be of interest to note here that at the request of pilots, all light-colored roadway surfaces in the vicinity of the North Island Naval Air Station, at San Diego, Calif., have been given a color coat of dark material.

The question of visibility at night is entirely one of adequate lighting. Wide markings on dark-colored pavements, or the outlining of such pavements in white, in conjunction with proper lighting equipment, would appear to provide the best visibility both for day and night flying.

A resilient surface aids materially in absorbing sound and impact, both of which are important considerations for airports. With these ideas in mind, it is not difficult to appreciate the trend toward the bituminous type of surfaces on the Pacific Coast.

John Nolen,[10] M. Am. Soc. C. E. (by letter).—In discussing this paper the writer would like to emphasize some of the points mentioned by Mr. Baker, but which owing to the broad scope of the subject had to be limited in consideration. These points are those having city planning relationship. As an outstanding feature of modern transportation the airport has an effect upon the city or urban community as a unit. To choose a site without consideration of all the elements of the community composition may mean that either the city may be injured by the location given over to the airport, or, in turn, the airport may not be so situated as to serve the city economically; or still worse, it may be so placed that it cannot develop business either from the city or serve as an adequate and safe stopping point on an airway for traffic from outside.

In these days American cities and communities of various types have come to a fairly complete comprehension of what city planning means. They have arrived at this comprehension through two channels: (*a*) The necessity of looking after the physical side of the city to meet the ever-increasing congestion due to the increased measure of transportation facilities, the growing density of population, and the recognized need for wider means of recreation; and (*b*) the increasing financial complications and problems that are thrust upon it by unplanned and misguided or wasteful developments of the past and present.

As a consequence of the growing population and this understanding of urban complications, the duly constituted authorities are preparing comprehensive plans to guide and control the various improvement activities of the city for the general welfare. These may be considered under three subdivisions, as follows:

(1) Thoroughfare plans which after careful study are prepared to designate the thoroughfare system and routes for the various classes of traffic; the most appropriate and economical widths of streets and pavements; the

---

[10] City Planner, Cambridge, Mass.

ways and means of effecting grade separations at conflicting traffic intersections; and related problems.

(2) Zone plans which divide the city into districts or groups of districts of like land use, so as to provide better for the general welfare and orderly growth of business, industry, and residence, and to secure permanence in the investment in land and buildings, and promote general welfare and public safety.

(3) Plans for parks and open spaces to secure the much needed recreational facilities for the present generation and provide for the future needs, as well as the acquisition and development of land to preserve features of natural scenic beauty and historic interest.

It is through these plans and their co-ordination as comprehensive planning for the future that provision should be made for the location of an airport, and no airport site should be selected without reference to these important controlling factors.

***Influence of the Thoroughfare System on the Airport Site.*—The outstanding contribution of aviation to modern transport is that of conservation of time, which Mr. Baker has rated as about 66⅔% saving for distances of 500 miles, or more, with a reduced advantage for shorter hauls. It is highly important, therefore, that the tremendous advantage in time saving be not diminished or lost through inadequate facilities on the ground between the airport and the points throughout the city or region from which air traffic will originate. It is the function of the thoroughfare plan to provide the best means of getting quickly and safely from one point in the community to another by creating direct routes, easy grades, and adequate width of pavement; by eliminating obstacles, such as railroad-grade crossings and highly congested street intersections; by the orderly control of traffic flow on highways; and by the provision of equally well-organized transit facilities.**

***Recognition of the Principle of Distribution of Land Uses, or Zoning in Relation to Airport Sites.*—An early recognition that airport location is one of the established land uses, such as business, industry, or residence, and that it is not an isolated item, will aid materially in locating the airport on a site where it may become a permanent asset to the community. To locate a use outside its proper zoning district makes of it a non-conforming use, injurious to the welfare of its neighbors and subject to many restrictions on expansion and development. The general terminal airport is an activity displaying characteristics which, in their preponderance, will eventually classify it as an industrial activity. It must provide storage in quite large proportions; it transacts business during the day and more and more in the future will do business at night; it has adjacent manufacturing, or perhaps will have manufacturing, facilities for ships or parts; it must provide repair facilities; and, in its fundamentals, it must combine the facilities of business and industrial uses. Specialized airports, of course, may be appropriate for residential districts, where the chief function is the embarking or debarking of passengers. In general, however, the airport is an industrial establishment and should seek its location in an industrial district. The related question of airways is important, and some of the principles involved in the selection**

of an airport should govern in the location of the airway over a community. As pointed out in the paper, it is important that the airport be so located as to reduce the actual flying over built-up sections and especially the residential areas.

Relationship of Recreational Facilities to Airports.—By appropriate design and co-ordination between the various agencies of the community government, the airport can be made a part of the recreational system of a community without in any way impairing its technical or operating efficiency.

The fundamental element in an airport is the landing field, about which are grouped other elements required for the complete functioning as a transportation agency. This landing field is an open space contrasting quite strikingly with the adjacent appendages. The airport in this open-space respect again contrasts with the built-up areas of the community, which, of course, vary in degree of density. It is, in part, this open area as one of the breathing spaces which makes it desirable to co-ordinate with the recreational system.

In addition to the benefit of the open space to the community, wherein there is free movement of air, there is always the value of the whole airport as an out-of-doors place of stimulating interest and activity, where imagination may be stirred and people aroused from routine ways by seeing and being associated with this modern system of transportation.

By skillful design the airport may be made a place of pleasure and amusement as a third form of recreational activity. Already one of the active pursuits in most airports is the sight-seeing trip which gives the visitor an opportunity to make his first flight or to cultivate a form of recreation for which a liking has already been established. Additional facilities for recreation would be established by the introduction of suitable restaurants, hotels, and allied places of amusement or sports.

Air transportation is making its way, and the city which does not adopt and plan measures for the airport will find itself in a few years left far behind in the relentless march of modern achievements.

S. N. Grimm,[11] Assoc. M. Am. Soc. C. E. (by letter).—The spectacular developments in design of large airships, both heavier and lighter than air, bring, as a result, a pronounced differentiation of air traffic. While the large airships tend to serve the same purposes as ground and sea common carriers, the smaller aircraft confine themselves more and more to functions similar to those of the automobile and motor-boat.

As time goes on great planes and dirigibles will be used for long-distance transportation along the principal routes of travel, with a limited number of stops, while the smaller planes will mostly fly comparatively short distances between the airports of a section of the country.

Many landing fields will serve exclusively the purposes of small aircraft, forming air highways, as it were. According to the data of the U. S. Department of Commerce, on July 1, 1929, there were 305 approved types of planes, 267, or 87%, of them having capacities of from one to six places. It is impossible to predict what will happen to this ratio in the future, but it is safe to assume that small aircraft will increase more rapidly in number than large airships. It is also evident that the future of private flying depends upon the improvement in design and in manufacturing of smaller planes, affecting their efficiency and cost, as much as upon the growth of the number of landing fields convenient for the purposes of private flying.

---

[11] Syracuse, N. Y.

The problem of locating such fields to be used by private planes for landing, storing, and minor repairs only, may be solved in the manner suggested by the author for major airports. However, the factors should be given different weights, according to the special purpose of the fields, the following being an example rather than suggestion:

Factors.	Weight.
(*a*) Topography	10
(*b*) Physical features	12
(*c*) Meteorology	12
(*d*) Location with respect to community served	5
(*e*) Transportation facilities	11
(*f*) Accessibility	25
(*g*) Legislative control	2
(*h*) Utilities available near site	3
(*i*) Cost	10
(*j*) Miscellaneous	10
Total	100

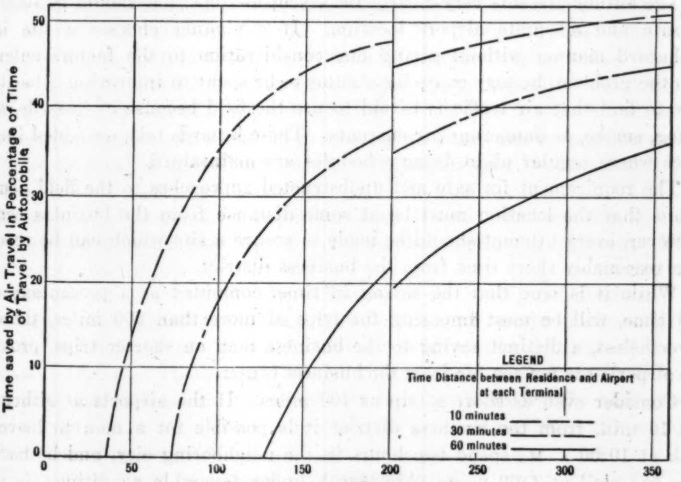


FIG. 3.—SAVING IN TIME ACCOMPLISHED THROUGH USE OF PRIVATE AIRPLANE INSTEAD OF AUTOMOBILE.

The weight of 25 points is given to the factor of accessibility of secondary airports, while the author gives to the same factor only 11 points for major airports. The reason for such a difference lies in the fact that, while the heavy air transport gains in its efficiency over ground common carriers in very long flights, the lighter air transport has to find its justification in an efficiency superior to that of highway travel in comparatively short distances.

Fig. 3, which is plotted in a manner similar to Fig. 2, shows a marked decrease in rates of time saved through the use of an airplane, instead of an automobile, due to remote location of airports, for distances up to 360 miles. The following assumptions are made in this connection:

Airplane speed, in miles per hour	80
Automobile speed, in miles per hour	33
Time allowed, in minutes, for getting a plane ready and placing it in the hangar	15
Time allowed, in minutes, for stop-over of plane and automobile after 200 miles	30
Time allowed, in hours, for stop-over of plane and automobile after more than 200 miles	10
Limit of distance traveled, in miles per day, by automobile.	360

The author is certainly right in his assertion that owners of private planes would not care to spend an hour traveling to the airport for a flight of like duration.

R. U. St. John,[12] Assoc. M. Am. Soc. C. E. (by letter).—The importance of applying sound engineering practice to the location of airports cannot be emphasized too strongly at the present time, when so many cities are contemplating such development.

The author presents very clearly the essential steps that should be followed in safe and adequate airport location. If a planner chooses a site in a haphazard manner without giving due consideration to the factors entering into the problem, he may cause large sums to be spent in improving a location, only to find that air traffic is afraid to use the field because of hazards, such as fog, smoke, or dangerous air currents. These hazards take on added importance where regular night-flying schedules are maintained.

The requirement for safe and unobstructed approaches to the field usually means that the location must be at some distance from the business center. However, every attempt should be made to secure a site which can be reached in a reasonably short time from the business district.

While it is true that the saving in time, computed as a percentage over rail time, will be most imposing for trips of more than 500 miles, there is, nevertheless, a distinct saving to the business man on shorter trips, provided the airports are not too far from the business center.

Consider even as short a trip as 100 miles. If the airports at either end are 15 min. from the business district it is possible for a man to leave his desk at 10:00 A. M., spend two hours in the neighboring city, and be back to sign his mail at 4:00 P. M. Air travel, under favorable conditions, is more restful than any other means of transportation and a man would return from such a trip without feeling fatigued. To make the same trip by rail or automobile would take the business man away from his desk at least eight hours and he would return tired and travel-worn. The saving of travel time, computed as a percentage, may not be great, but the saving of business hours is of real value.

---

[12] Mgr., San Francisco Bay Airdrome, Alameda, Calif.

The study of meteorological conditions is of extreme importance in airport location. No one would think of building dock facilities for large boats drawing 30 ft. of water without first determining, by soundings, that the harbor would handle craft of that draft. Many communities, on the other hand, proceed with airport development, at considerable expense, without adequate data on wind direction and velocity.

The two most important meteorological factors from the aviator's standpoint are fog and wind direction and velocity. There is a real need for simple and inexpensive instruments that will record these data. If such instruments were available they could be installed for a period of six months or a year at proposed locations and would thus furnish the information accurately. They would be particularly valuable to smaller communities that do not have Weather Bureau Stations.

So far as the writer knows, there is no way of recording the actual presence of fog accurately other than by visual observation. In a recent study, the Airport Commission of the City of Sacramento, Calif., made use of recording hygro-thermographs in order to obtain data on fog conditions. These instruments were set up at representative stations throughout the area being studied. They furnished a continuous record of humidity and temperature during the three months' period when fogs occur in the region of Sacramento. By using the percentage of time during which 100% humidity was recorded, it was possible to rate the stations in the order of their fog-forming tendencies. These results seemed to check fairly well with simultaneous visual observations; in other words, the stations where 100% humidity occurred for the greater proportion of time, were those at which the fog was most frequent and most persistent. It would be interesting to know whether similar results would be obtained in another region where the topography is uneven and where the fog belts are more sharply defined.

There are instruments for recording wind direction and velocity, but they are expensive to install and require considerable attention to maintain. These instruments record all air movements from a zephyr to a hurricane. For airport work the direction of light winds, up to about 10 miles per hour, is of little importance, because ships can land "cross wind" under such conditions without hazard; but the direction and velocity of the stronger winds are important factors. There would seem to be no serious mechanical difficulty in turning out an inexpensive instrument that would record the higher wind velocities and show their direction within 10°, which is sufficiently accurate for locating runways.

The factors used as a basis of comparison of different sites should be definite and specific so that their value can be obtained by computation, as far as possible. The choice of a municipal airport location is a problem that usually interests real estate men with property to sell. If the factors used in determining the comparative merit of the different sites are assigned values on the basis of judgment alone there is opportunity for endless discussion and a danger perhaps of setting aside the recommendation of the body making the report. On the other hand, if the majority of rating factors are computed there is little room for argument.

As an example of a set of rating factors, most of which can be computed, Table 2 is taken from the report of the Airport Commission of the City of Sacramento.

TABLE 2.—RELATIVE WEIGHT OF FACTORS INVOLVED IN CHOOSING THE LOCATION OF AN AIRPORT AT SACRAMENTO, CALIF.

Item No. (1)	Subject. (2)	Rating factors, ideal site. (3)	Rating factors, site selected. (4)
1	Freedom from dense river fogs	15	12.3
2	Freedom from bad air currents	10	8.0
3	Area of field	10	9.2
4	Configuration of field	5	5.0
5	Approaches, satisfactory areas under take-off	8	8.0
6	Favorable prevailing winds relative to runways	5	3.0
7	Prospective neighborhood development	4	3.0
8	Possibility of expansion	4	4.0
9	Location relative to axis of air travel	10	6.0
10	Distance from post-office	10	5.0
11	Distance from population center	10	6.3
12	Distance from railroad	4	2.0
13	Distance from airplane factory sites	5	2.0
	Total	100	73.8

This Commission spent nearly a year studying twelve proposed sites on a basis similar to that which the author of this paper advocates and was successful in securing the adoption of its report. The Sacramento Municipal Airport is now being constructed on the site recommended.

After assigning factors for the ideal site, the next step was to assign the proper values to each rating item for the various sites under consideration. The total of these values for any field would be its rating factor (see Column (4), Table 2). The assignment of the values was made by computation as far as possible, although a few of the items were necessarily matters of judgment only.

1.—Freedom from Dense River Fogs.—The meteorological survey gave the information from which values of Item No. 1 were obtained. The four meteorological stations were rated in the order of their freedom from fog conditions, and this information was applied to the sites in the region adjacent to each station.

2.—Freedom from Bad Air Currents.—Values of Item No. 2 were matters of judgment and were assigned after numerous flights over the sites to determine the presence of any unusual air conditions. The proximity of trees or buildings causes uneven air currents and this fact, was also used in determining this factor.

3.—Area of Field.—It was decided that an area of 250 acres represents the ideal site because it would give all the space needed at present and allow for a moderate expansion program. Sites larger or smaller than this ideal size were penalized in proportion. For example, the rating factor for a 400-acre tract would be $10 \times \frac{250}{400} = 6.25$.

4.—Configuration of Field.—The factor assigned to Item No. 4 was largely a matter of judgment. A rectangular field at least 2 500 ft. wide, with its length in the direction of the prevailing winds, would be the ideal shape. Odd-shaped fields on which it was difficult to lay out proper runways were penalized.

5.—Approaches, Satisfactory Areas Under Take-Off.—The total length of runways in the north and south and northwest and southeast directions was used as the basis for rating Item No. 5. If these lengths totaled 8 000 ft., the site received a full rating of 5. Shorter lengths were reduced in proportion.

7.—Prospective Neighborhood Development.—Factors for Item No. 7 were matters of judgment. If the trend was toward commercial or residential building, a low rating was given, as this type of development close to an airport is hazardous. If the trend was only agricultural development, a high rating was given.

8.—Possibility of Expansion.—If all four sides of a tract were open to expansion, the full rating of 4 was given. If one side was blocked by a railroad or similar obstacle, the rating would be 3, etc.

9.—Location Relative to Axis of Air Travel.—Item No. 9 was appraised largely on the basis of judgment and conjecture as to future lines of air travel. In general, sites to the east of the town received the higher ratings because they are nearer the San Francisco-Reno Air Line.

10.—Distance from Post Office.—For any field, the rating for Item No. 10 was determined by the formula:

$$P = 10 - d \qquad (1)$$

in which, P is the rating factor, and d is the distance, in miles, to the nearest post office. Ten was used as a limiting factor because it was felt that a site more than ten miles from the Post Office would have no value as an air mail field when so many closer sites are available.

11.—Distance from Population Center.—The value for Item No. 11 was computed the same as that for Item No. 10.

12.—Distance from Railroad.—Sites more than a mile from a railroad received zero rating. Those less than a mile were rated in proportion, the one having full rail connections adjoining the property receiving a full rating of 4.

13.—Distance from Airplane Factory Sites.—In general, Item No. 13 is affected by the same factors as Item No. 12, but special recognition was given to sites that had particularly good factory locations adjoining.

The data finally to be considered by the Commission consisted of the estimated cost of improving each one of the twelve proposed sites and a rating factor that indicated the value of the site as an airport. The last step was to divide the total estimated cost for each site by its rating factor to get a unit rating cost. This indicates the site which has the lowest cost per unit of rating and is a valuable indication of the best site from the standpoint of both rating value and cost.

It is hoped that one result of this paper will be to establish the general principles of airport location and to pave the way for a uniform rating schedule that will facilitate the comparison of proposed sites as well as completed airports.

John F. Partridge,[13] M. Am. Soc. C. E. (by letter).—The writer wishes to offer two suggestions which might be overlooked amid the many factors that will confront location engineers on airport developments.

The location of the airport for the small town should be made in the interests of the through transportation of mail, express, or passengers. The volume of business having the small town as its terminus is generally small by comparison, and time saving between the larger centers is the prime consideration. The smaller center will generously concede priority of convenience for the privilege of a tie to its larger neighbor. Transportation between the individual smaller centers on the route of flight is a result of convenience generally due to a scarcity of other available means of transport, and airport location should be again made in the interests of through service, either present, or possibly in the future. Utter disregard, by through service, of the demands of the smaller communities, however, is not in the best interests of air transportation. The cultivation of public interest in aviation in the smaller town will result in increased air travel with its inevitable concentration around the larger communities.

Airport location in mountainous country should include consideration of the angle of slope from the tops of the neighboring mountains into and from the landing field. For smaller towns where through-transport planes stop for refueling, or the picking up of mail or passengers, it should be remembered that circuitous climbing to gain the necessary elevation to cross the mountains, may consume a considerable part of the total time between terminals. For larger centers, while this question of time lost in gaining altitude is still an important consideration, the other factors usually peculiar to a proper location of the airport may have to take precedence.

William J. Fox,[14] Assoc. M. Am. Soc. C. E. (by letter).—Air transportation has so fixed itself into the modern method of traveling that it is now an established economic necessity.

The time element factor which differentiates the automobile from the airplane is so great that, economically, the business man can no longer afford to travel long distances by automobile. The time element in modern business is of such importance that this one factor alone determines the success or failure of the business structure. To no small degree has this same element entered into present-day social and recreational existence.

The fact that transportation is necessary at all is, basically, a handicap to any business. As an example, two businesses of identical character, one requiring transportation facilities for its operation and the other so located as to not require such facilities, other things being equal, it will be found that the transportation factor determines, in terms of time and cost, the financial differential of the two enterprises. Therefore, the form of transportation which minimizes most successfully the time element, with a corresponding reduction of cost, and which more closely approximates the condition in which no transportation is necessary, is of the greatest economic value. Aviation or air transportation is and no doubt always will be the most effective in meeting the foregoing requirements.

---

[13] Hydr. Engr., Empresas Electricas Brazileiras, S. A., Rio de Janeiro, Brazil.

[14] Chf. Engr., Los Angeles County Regional Planning Comm., Los Angeles, Calif.

Safety in Aviation.—Taken in relation to certain facts, the present mortality arising from air travel is surprisingly small. The vital statistics of the United States, England, and Wales indicate that the number of deaths in aviation per 1 000 000 of population is quite uniform.

The airplane, in the main, has gone through the throes of the experimental stage in so far as structural characteristics are concerned. Airplane motors have been perfected to the same degree as the most finely finished watch. Accidents due to structural defects are an extreme rarity. Only 5% of the total number is due to this cause. The fact seems to be that passengers, on regular air lines owning perfect machines that are under the control of regularly licensed pilots, enjoy a good degree of safety.

The airplane undoubtedly has achieved the highest place as a medium for transportation, especially for long distances. The financial advantage of utilizing this form of transportation for both passengers and light cargoes is strongly manifested by the air transportation companies that have been formed during the past few years and by the tremendous amount of capital being invested in the aviation industry.

TABLE 3.—COMPARATIVE MORTALITY IN AIR TRAVEL.

	Year.	Mileage.	Fatalities.	Average per 1 000 000 population.
United States	1926	....	160	1.5
United States	1927	....	214	1.6
United States (first six months)	1928	....	161	1.6
British Air Lines	1925-1927	2 471 000	None	...
British Air Lines	1922-1924	2 596 000	15	1.4
Luft Hansa, European	1925-1927	12 605 000	22	1.7
United States	1921-1926	....	908	1.5
England and Wales	1921-1926	....	265	1.1

The success of the airplane as a safe and efficient means of transportation is largely dependent upon the selection and design of the airport on which it must land and take off. Airport location looms as one of the chief factors involving safety as well as the future development of aviation. The present airports in American cities in general, even those of the largest size, are being adjudged as inadequate and inefficient.

Selection of Modern Airports.—To gain public confidence in this form of transportation, airport sites must be selected on the basis of engineering judgment coupled with the advice of flying experts. They should be laid out to the best advantage, with the aid of engineering skill.

If aviation is to grow as it has a right to grow, it must enlist the services of the flyer himself, the civil engineer, the architect, and the city planner, as it has enlisted the services of the scientists, the navigator, and the motor expert.

One or two serious accidents to passenger planes, due to the unwise selection or faulty design of an airport, has a disastrous effect upon the aviation industry as a whole. The right to fly air mail over a community might even be put in jeopardy. In view of the position that aviation now holds in the economic structure of the United States, it is essential that public confidence should be augmented rather than impaired.

The selection of a field is more than a real estate transaction. Competent men who are engaged in the actual flying of aircraft should be consulted in order that the particular field under study be given the official endorsement of those men who will be called upon to fly planes to and from the airport when selected.

Technical Advice.—Mr. Baker emphasizes the importance of certain factors in determining the proper location of airports. It is imperative that factors be used which will tend to give an impartial rating by a purely mathematical or mechanical process. These factors should be a minimum in number, but at the same time representative of the vital characteristics of a safe and efficient airport. Quite as important as the factors themselves is the procedure followed in leading up to their application, for example: (*a*) Survey of the area to be served; (*b*) technical advice; (*c*) future predictions of population and airplane growths; (*d*) economic analyses; and (*e*) application of factors by qualified experts.

The lack of technical advice is not only expensive in the initial selection of the field, but poor judgment in the selection as to proper location usually entails great cost in order to modify conditions to make the airport safe and adequate. The cost of adjusting topographical conditions, the elimination of hazards, and the installation of drainage systems on a poorly selected field runs up the cost to such an extent that it might have been more profitable to have selected a site, the initial cost of which was far greater. Likewise, the adequate laying out of the development plan for an airport is, in many respects, similar to planning a small city.

The matter of controlling the height of buildings constitutes a strong factor. Height limit regulations in the vicinity of a field give the operators assurance that they will have the best possible use of their field; they also contribute added safety to those using the air as a means of transportation.

Selection with Respect to Growth Trends.—Airport selection and construction require a careful study of land areas, growing and shifting population, industrial and residential development.

The selection of an airport site is properly a part of city planning, and as such should be in conformity with the future development program. As part of its planning activities, every city should prepare an airport development program that will provide for future needs on the basis of real economy.

The airport is the gateway to the modern city. If properly selected, a satisfactory financial return can be shown for the investment involved. To make the investment economically sound, however, the following principles must be rigidly adhered to:

1.—The site should be selected by qualified experts.
2.—The land should be owned in fee or on long-term lease.
3.—The airport should be accessible to those served by a maximum of 20 min. surface transportation time.
4.—The field should be of the highest rating commensurate with use.
5.—Proper legislative control should be enforced over surrounding development.
6.—The field should be properly related to the community served and other fields.

Los Angeles County Method of Locating Airports.—In the latter part of 1927 Los Angeles County made the study of airports, landing fields, and various ground requirements a part of the activities of the Regional Planning Commission. This Commission is a part of the County Government and is charged with the duty of planning the wholesome development of the County as one great commonwealth. The area involved was slightly less than 5 000 sq. miles, including forty-four incorporated cities and thirty-six unincorporated towns. It was obvious that thorough planning had to embrace the requirements for this new form of transportation. Otherwise, any shortcomings would be discovered too late to adjust the development economically to the situation.

Population and Airplane Density.—Fig. 4 shows the increase in the number of airplanes in Los Angeles County, California. It likewise shows the growth of population for a considerable period. Both these growth curves are projected into the future for a 30-year period. From the resultant quantities of these two curves, a third curve is plotted, called "The Airplane Density Curve", which represents the number of persons per airplane as of the past and as anticipated in the future.

The present status in this County (1930) is 1 airplane to every 2 500 persons. It is estimated that by 1980 this ratio will be 500 persons per airplane, and in the year 2030, 100 persons per airplane.

With the population curve as the basis and the 20 min. of surface transportation as a maximum figure, some basic information is provided upon which to calculate the size and frequency of the airports that will be necessary to satisfy future needs.

Due to the very rapid growth experienced, it was evident that the improper location of one road might forever destroy a dominant site for a future airport or landing field. The time to locate airports is before land is subdivided or cut up by streets or highways. Therefore, a study of the requirements that would be needed for the future of aviation was made in conjunction with the plans and studies for highways, population densities and trends, industrial development, transportation facilities, zoning, and land subdivision. The results have been co-ordinated into a master plan. As divisions of the Master Plan are completed they are submitted for official and public approval. The entire procedure involves six steps, namely, (1) to take an inventory of existing conditions; (2) to form an aeronautical advisory committee; (3) to make a comprehensive study of available sites; (4) to estimate the cost of equipment and improvements for airport operation; (5) to analyze the financing and probable earning ability; and (6) to design the airport.

Step No. 1.—Inventory of Existing Conditions.—A survey and inventory were made of all existing airports, landing fields, and aviation facilities within the County. With respect to each existing airport information was obtained as to the following factors:

(1) Location.
(2) Description.
(3) Runways and landing areas.
(4) Obstructions and hazards.
(5) Equipment.
(6) Improvements.
(7) Conditions of ownership.
(8) Class.
(9) Area.
(10) Elevation.
(11) Drainage conditions.
(12) Soil conditions.
(13) Meteorological data.
(14) Transportation facilities.
(15) The name of the operator.

The survey revealed that there were fifty-two fields in operation. They were of all sizes, shapes, and conditions of ownership. The relative sizes were found to be as follows:

Size, in acres.	No.
20 to 30	15
30 to 50	6
50 to 100	7
100 to 160	7
250	1
380	1
640	1
Not determined	14
Total	52

The acreage was variously owned, as shown in Table 4.

TABLE 4.—COMPARISON SHOWING NATURE OF OWNERSHIP.

Nature of ownership.	Number of acres.	Number of fields.
Public	784	5
Private	570	8
Leased	227	39
Not classified	2 050	
Total	3 631	52

Many of the fields were operating on leases of from thirty days to ten years, as shown in the following list:

Length of lease.	Number of airports.
30 days	20
60 days	2
90 days	3
1 year	2
2 years	1
3 years	4
5 years	4
10 years	3
Not leased	13
Total	52

This short-term lease condition greatly handicaps the developers and operators in securing loans or encouraging investments for expansion or permanent improvements.

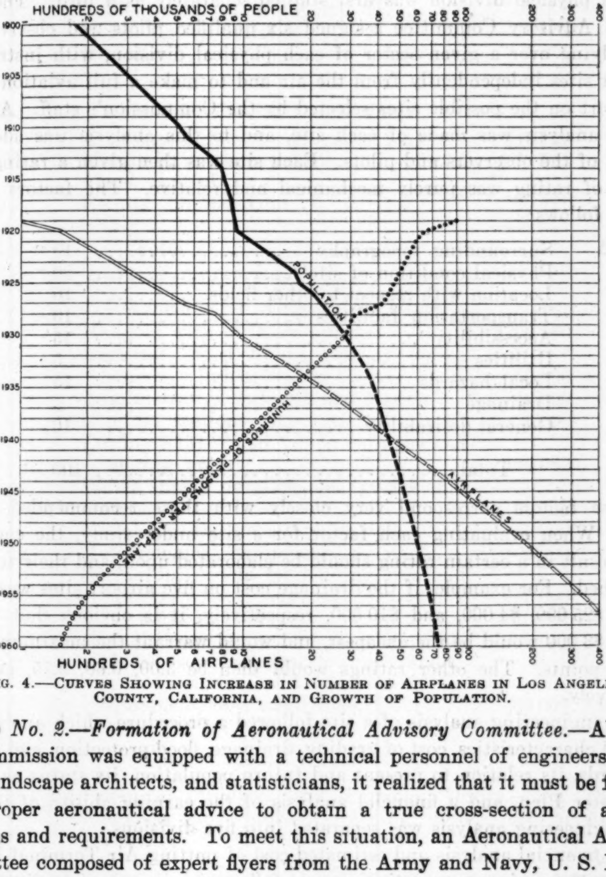


FIG. 4.—CURVES SHOWING INCREASE IN NUMBER OF AIRPLANES IN LOS ANGELES COUNTY, CALIFORNIA, AND GROWTH OF POPULATION.

Step No. 2.—Formation of Aeronautical Advisory Committee.—Although the Commission was equipped with a technical personnel of engineers, architects, landscape architects, and statisticians, it realized that it must be fortified with proper aeronautical advice to obtain a true cross-section of aviation problems and requirements. To meet this situation, an Aeronautical Advisory Committee composed of expert flyers from the Army and Navy, U. S. Department of Commerce, air transport companies, aerial police, and civilian flyers, was formed. The studies, designs, and accomplishments of the Commission were placed upon a basis as nearly scientific as possible.

Step No. 3.—Comprehensive Planning of Airports.—After all the existing fields had been surveyed, plotted on maps, and evaluated, the technical staff of the Commission divided the County, for the purpose of study, into the following natural physical divisions: (*a*) Antelope Valley; (*b*) San Gabriel Valley; (*c*) San Fernando Valley; (*d*) La Crescenta Valley; (*e*) the Central Area of Los Angeles; (*f*) Southeast Section; and (*g*) Pacific Coast Beach Section.

With the aid of maps showing the existing and proposed highway systems, other maps showing land that was subdivided, and with U. S. Geological

Survey maps showing topography (scale 1 in. = 2 000 ft., with 5-ft. contours), the technical staff indicated every flat parcel of land that had any possibilities whatsoever as a potential airport.

Each physical division was first studied separately as a unit. The Aeronautical Advisory Committee assigned six qualified pilots and observers the job of flying over a given sector of each physical division, with instructions to select sites independently from the air and to make a full aviation survey and report on the possible sites selected by the Commission's staff. An engineering analysis was made of each site, and to this analysis was added the findings of the observers and pilots. Each site was then given a rating. The process of rating was purely mechanical and relative. The factors applied were as follows:

Surrounding topography	10
Physical condition of site[15]	20
Location with respect to other fields	10
Transportation facilities	10
Accessibility	15
Utilities	5
Local hazards	15
Drainage	5
General desirability	10
Total	100

These factors correspond very closely with those recommended by Mr. Baker. When evaluating each factor for a site under study, the conditions which result in a certain rating should be elaborated upon, and their relativity established. For example, if the drainage cost on five airport sites was $3 000, $5 000, $25 000, $4 000, and $10 000, respectively, it is obvious that the one costing $3 000 would be the cheapest, and would warrant the maximum rating of 5.00 points. The other ratings would then be 3.00, 0.60, 3.75, and 1.50, respectively.

The engineering analysis of a site followed a procedure which analyzed the physical characteristics, cost of grading, drainage, flood protection, and removal of hazards, its relation to present and future population, its status as part of the Master Plan, and a financial analysis of the earning ability of the field. This engineering analysis was separated into five divisions.

The financial analysis and estimated cost of putting Air Terminal No. 8 in condition[16] for airport operation, was as follows:

Division 1.—Physical Analysis of Site:

		Percentage of total.
Total area of site, in acres	968	
Total acres that can be used for airport purposes if improvements in Division 2 are carried out	935.73	96.67
Acres subject to overflow of river and washes	32.27	3.33
Area (acres) unusable for airport purposes	32.27	3.33

---

[15] Meteorological study was over the area as a whole.

[16] Does not include cost of land.

Division 2.—Estimated Cost of Improving Site:

Removing 877 acres of walnut trees, buildings, irrigation lines, etc.	$40 000	
Grading (west of flood control levee) 100 000 cu. yd. of cut and fill	50 000	
29 000 lin. ft. of fence	29 000	
Drainage	5 000	$124 000

Division 3.—Outstanding Bonded Indebtedness:

El Monte School	$16 629	
Mountain Avenue School	342	
Flood control	5 351	
County improvement	19 722	$42 044
Total		$166 044

Division 4.—Estimated Cost of Equipment and Improvements for Airport Operation:

Hangars with concrete aprons	$200 000	
Lighting (flood lights, runway lights, boundary lights, beacon light)	50 000	
Administration building and depot	40 000	
Comfort stations	3 000	
Turfing 2 151 364 sq. yd. on runways and landing area	129 082	
Water supply development (pumps and storage)	50 000	
Telephone and telegraph system	15 000	
Meteorological equipment	5 000	
Sprinkling system for landing area and runways	35 000	
Road work	10 000	
Miscellaneous equipment (trucks, roller, blades, scarifier, tractor unit, ambulance and first aid equipment, fire equipment, and small tools)	30 000	$567 082
Total		$733 126

Division 5.—Financial Analysis and Earning Ability.—The study of finances and probable earning ability are arranged in Table 5.

Summary of Financial Analysis.—The foregoing items may be summarized as follows:

(A) Annual Earnings for 50-Year Period, 1930-1980:

Net annual profit on walnuts = $82.24 per acre (average) × 300 acres	$24 672	
Average annual earnings over 50-year period.		$275 460
Average annual cost of operation and maintenance	$30 000	
Annual interest on $733 126 (improvements) plus $2 500 000 (land) at 6%	193 987	223 987
Average annual net profit (50-year period)		$51 473

(B) Ultimate Annual Earning Capacity of Site No. 8:

Anticipated ultimate population, San Gabriel Valley (Section 2-E)	3 000 000	
Anticipated airplanes in 1965, San Gabriel Valley	6 000	
Anticipated airplanes when San Gabriel Valley reaches 3 000 000 population	30 000	
174 acres of hangar space at 3 cents per ft. per year		$227 380
Depot and administration office space rental, 50 000 ft. at $1.00		50 000
100 acres available for shops, accessory houses, manufacturing plants, etc., at 10 cents per acre		435 600
Expected annual revenue from concessions, landing rentals (student pilots), lockers, transport passenger tax and miscellaneous		50 000
Ultimate annual earnings		$762 980

As a result of this study, Air Terminal No. 8 was given the following rating:

Surrounding topography	10
Physical condition of site	20
Location with respect to other fields	10
Accessibility	13
Transportation facilities	10
Utilities	3
Drainage	5
General desirability	10
Local hazards	12
Total	93

TABLE 5.—FINANCIAL ANALYSIS AND EARNING ABILITY, AIR TERMINAL NO. 8.

	1930	1940	1950	1960	1970	1980
Population (San Gabriel Valley)	250 000	480 000	625 000	750 000	900 000	1 050 000
Number of airplanes (San Gabriel Valley)	62	480	2 080	5 000	8 500	10 000
Number of airplanes at terminal	31	240	1 040	2 500	4 750	5 000
Number of hangars required	2	16	69	166	316	334
Square feet of hangar space	40 000	320 000	1 380 000	3 320 000	6 320 000	6 680 000
Rental from hangar space	$ 1 200	$ 9 600	$ 82 800	$139 200	$259 200	$400 800
Revenue from manufacturing, accessories, etc	2 000	8 000	20 000	40 000	80 000	160 000
Revenue from taxi schools, landings, transports, etc	300	2 400	20 700	25 000	27 000	26 000
Revenue from utility service	500	3 000	5 000	7 000	9 000	11 000
Revenue from office service	500	5 000	12 000	20 000	30 000	40 000
Miscellaneous revenue	100	500	2 000	5 000	8 000	10 000
Total revenue	$ 4 600	$28 500	$142 500	$236 200	$418 200	$647 800
Net profit from walnut crop	$24 672	$24 672	$ 24 672	....	....	....
Gross annual earnings	$29 272	$42 172	$167 172	$236 200	$418 200	$647 800

Analyses and studies similar to the foregoing were made for each site. Then the best were selected, by the process of elimination, to serve the County in a comprehensive way with airport facilities. The comprehensive plan was so designed and arranged that the existing airports which had received a satisfactory rating were properly recognized. The proposed airports were so arranged and situated as to serve the ultimate population and were so chosen that the development of the sites themselves could be accomplished if and when they were in demand. The official highway plan of the County was adjusted to conform to and preserve both the existing and the proposed airports.

The general procedure was to select, at a strategic location in each physical division of the County, a major air terminal, from which would operate transport planes, cargo planes, and all large aircraft.

Definition of Major Air Terminal.—A major air terminal is an airport of sufficient size to permit the safe and efficient operation of the largest transport planes that are now in service or likely to be placed in service. It must have a runway of at least 5 000 ft. in length and 400 ft. in width, or must be so equipped with apparatus or mechanism as to permit the taking off of planes in like number and with the same degree of safety as would the runway stipulated. It must also be able to provide a separate area that will permit landings in all directions. The dimensions of such landing area must be at least 2 500 by 2 500 ft., or must be equipped with apparatus or mechanism to accomplish the same effect in the matter of safety and number of landings as would the area stipulated.

A major air terminal must have sufficient area to enable the erection of storage hangars for at least 1 000 airplanes; it must have adequate space for shops, manufacturing plants, and a passenger and freight depot; and it must be adequately served by both rail and highway transportation facilities. It must also be so located as to serve, adequately, a large population, either actual or potential. Furthermore, it must be able to warrant the highest class airport rating of the Aviation Division of the Department of Commerce.

In each physical division of the Los Angeles County survey there were selected satellite fields to serve each community. Private ships, police planes, Army and Navy ships, taxi planes, and small craft will operate from these fields.

Definition of Satellite Airport.—A satellite airport is a field so designed as to accommodate ships smaller and of less wing spread than transport planes. It must be of sufficient size to permit the construction of a runway at least 1 800 ft. in length. It must also have hangar space for the storage of airplanes, such space to be commensurate with the population served.

The general purpose of a satellite field is to serve the owners of private planes, commercial taxi planes, light military planes, and police planes; also, to serve as emergency landing fields for larger planes in distress.

The satellite fields in Los Angeles County were so located as to be accessible by ground transportation not involving more than 10 min. of time from any point. In addition to serving the community with airport facilities for the present and future, in so far as private ships are concerned, these satellite airports give opportunity for aviation facilities for long-distance travel. This is accomplished by the operation of taxi planes from the satellite fields to the major air terminal, from which operate the large passenger ships.

The distribution of air mail to local points is accomplished in a similar manner.

Fig. 5 illustrates the comprehensive airport development plan for Los Angeles County. As each physical division of the County is surveyed and reported on by the Commission's technical staff and its Aeronautical Advisory Committee, detailed plans are prepared for the design and development of each recommended site.

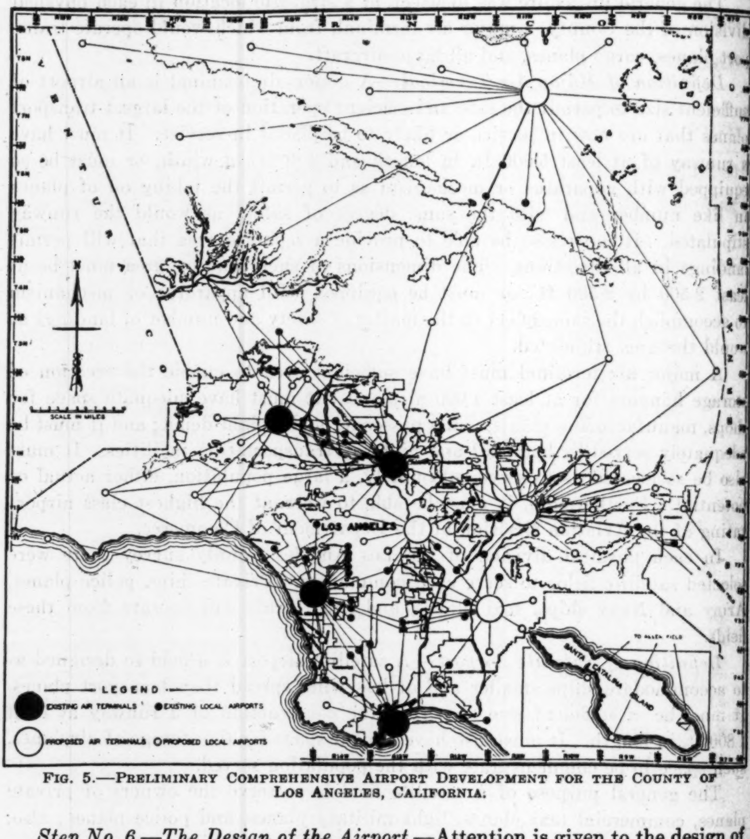

SCALE IN MILES
LOS ANGELES
SANTA CATALINA ISLAND
LEGEND
EXISTING AIR TERMINALS
EXISTING LOCAL AIRPORTS
PROPOSED AIR TERMINALS
PROPOSED LOCAL AIRPORTS

FIG. 5.—PRELIMINARY COMPREHENSIVE AIRPORT DEVELOPMENT FOR THE COUNTY OF LOS ANGELES, CALIFORNIA.

Step No. 6.—The Design of the Airport.—Attention is given to the design of the major air terminal for the reason that the area involved and the design of the field itself must be commensurate with the population served. The future population density gives a fair basis for the facilities required. At the outset it is recognized that the major air terminals will have the greatest concentration of aircraft operation. The area of the immediate physical division that each major air terminal serves ranges from 200 to 300 sq. miles and an ultimate population of from 1 000 000 to 3 000 000.

One of the chief factors in the safety of the airplane as a means of transportation is the proper and efficient design of the airport. In the design of all major air terminals one of the basic principles that has been followed in the plan is to provide for the segregation of the landing area and the runways for taking off. Another is to have runways of sufficient length to permit landings on the field in the event of a "dead stick" or a "concked motor". It has been quite definitely established that these basic principles contribute to greater safety and efficiency. Such design will permit the simultaneous landing and taking off of a squadron of planes; it likewise permits the continuous landing or the continuous taking off of ships, on schedule or off schedule, with a large factor of safety and efficiency.

Application of the Comprehensive Plan.—It is generally recognized by those vitally interested in the future of aviation that it is essential that a systematic and comprehensive plan be followed. Those developing airports in Los Angeles County have the benefit of the impartial study and advice of the Regional Planning Commission and its Aeronautical Advisory Committee; and the selection of major air terminals or local fields, whether private or public, has closely followed the recommendations as set forth in the Master Plan.

The selection of a site in accord with the plan gives protection in the form of zoning; it provides for the proper construction and location of ground-transportation facilities, regulation of the height of structures in the vicinity of the field; and also guards against having the field declared a nuisance.

The financial set-up for the development of air terminals differs somewhat in each instance. If the purchase of any air terminal is to be made from public funds, it is the intention of the Regional Planning Commission of Los Angeles County that such purchase shall be made on the basis of its contribution as a part of the "Comprehensive Airport Development Plan".

Summary.—In stipulating the factors which have to do with the governing of airport locations, whether for large fields or small, it is essential that certain fundamental principles in city planning be recognized. Some of the important factors to be considered are those that have to do with the future growth and character of the community which the airport is to serve, always remembering that aviation is still in its infancy. The procedure being followed in Los Angeles County and the factors used have accomplished very satisfactory results.

When speed is considered, the element of safety plays a predominant part. The more speed that is acquired, the more safety is sacrificed. Travel by air will never be as safe as travel on the ground. Much can be done, however, to make air travel reasonably safe. The index of the safety factor now rests with the design, location, and adequacy of airport facilities. The majority of airplane accidents occur either when landing or taking off. If the same factor of safety can be incorporated in the design and location of airports as is accomplished in the design and construction of the plane itself and by the thoroughness of the pilots, aviation will indeed be a reasonably safe means of travel.

Perry A. Fellows,[17] M. Am. Soc. C. E. (by letter).—This paper emphasizes some of the points that the writer has stated[18] on other occasions. The methods outlined have been used in a number of instances as an aid in selection of airport sites.[19]

There are two limitations that should be definitely borne in mind whenever an attempt is made to codify characteristics: First, such a method is a tool and should be used honestly for the reason that, just as with any other tool in the hands of those who so wish, it can be misused; and, second, the group best suited to make use of such a tool is the engineering group.

If these two principles are kept uppermost in mind the use of airport characteristics will avoid the criticism that otherwise may be met. There are people who oppose setting up such a classification as that recommended in the paper; especially do they oppose it on the grounds that it is possible for some one to believe that a way is thus made easy for the layman to select the best possible airport site. Such criticism can always be met with the cautionary advice that the instrument should be used only in its proper field and that its use should be entrusted only to the competent and qualified engineer.

W. W. Crosby,[20] M. Am. Soc. C. E. (by letter).—To one who has been familiar with the development of County, State, and Federal Highway systems during the past thirty years or more, this paper has suggested some analogies of interest. Both airways, with their airports, and highways, with their foci or termini, are transportation systems. The locomotion may seem quite different, but certain principles are common, and airships do "land" even though, as the author suggests, air and water navigation have much in common.

When the highway systems were first started, every little village fought strenuously to have the improved roads run through it and along its main street. As the expansion of the systems took place, and such transcontinental routes as the "Lincoln Highway" were conceived, officials in the larger cities also fought for the routes to run through their territory.

A few years ago a reaction took place. The same places began to appeal piteously to the highway authorities for the removal of the through traffic to by-passing routes so that the localities might be relieved from the serious dangers and damages caused to the local use of the streets by the through traffic; and a general movement along this line is already under way.

It seems to the writer that analogies accompany these facts; and he believes that they should be seriously considered in connection with the problems of airport location.

In this same connection a detail of airport construction may have some weight. May not the developments in airport surfacing have some effect on location just as the developments in highway surfaces have affected highway location?

---

[17] City Engr., Detroit, Mich.

[18] *American City Magazine*, May, 1928, and a more complete discussion in the Associated Technical Societies *Bulletin*, April, 1928, reprinted in *Engineering News-Record*, September, 1928.

[19] "Standards and Score Card for Municipal Airports," by N. L. Engelhardt, Jr., *Airway Age*, October, November, and December, 1928.

[20] Cons. Engr., Coronado, Calif.

Formerly, many conflicting factors influenced the type of surface necessary on highways. For example, smooth, hard surfaces were not suitable for slow, horse-drawn traffic and vehicles with metal tires that ground the road surfaces into dust. As the motor traffic has become predominant on many systems of highways it has simplified the problems of surfacing by removing these conflicting factors, so that the traffic is composed only of soft-tired vehicles which require smoothness and which create no detritus. Curves are required to be flatter, visibilities must be longer, and grades may be steeper; but alignment should be more direct and widths must be greater. To give general satisfaction surfaces may be less varied and more "standard".

May not some of the developments in air vehicles, that now seem quite probable to threaten, similarly affect the decisions as to airport construction and location? For instance, suppose that a mechanical modification in the wing of a plane should be developed so that a plane could check its horizontal speed and drop gently, and almost vertically, to its landing. Would not the allowance for "gliding angle" in airport location be reduced? Similarly, if the airplane could be checked in its forward movement, would not the necessity of depending on the friction of the tires or the tail-skid for this result be relieved so that the surfacing of the landing field could be simplified and improved? Possibly, also, the rising angle of a starting plane could again be increased, and the location again affected. These results might affect the size of the area required and thus enable locations to be made advantageously where now they seem impossible.

Furthermore, if an airport may be regarded as something distinct from an airfield, may not the location problems of the former be simplified? Why should not the housing, repairing, fueling, oiling, tuning-up, and such work go on in convenient but separated "yards" or "docks" (as in the case of railroads and shipping lines), so that the ports or depots might be located with the convenience to the passengers that is necessary for commercial popularity? Certainly training or practice activities are not necessary functions of airports.

Even in the new art of airway engineering, it seems to the writer, after considering the many mistakes that have been proved on him as well as on others in years of highway engineering, the saying that "the wise man learns from the experience of others; any one can learn from his own", is still worth remembering.

E. K. SMITH,[21] ASSOC. M. AM. SOC. C. E. (by letter).—The considerable increase in air passenger traffic, quite largely due to the lowered passenger rates, and the increase expected in express service should cause special importance to be placed on certain of the factors in airport location covered in Mr. Baker's paper. These are, particularly, the physical features, the accessibility to the community served, and the ground transport facilities to and from the airport. For mail service alone facilities for transport to and from the airport will not be so important. It is probable that within a few years all major airports will have pneumatic tube service from the main post office.

---

[21] Asst. Mgr., Highways and Municipal Bureau, Portland Cement Assoc., Chicago, Ill.

For a purely express or mail business the surroundings, appearance of the field, etc., do not play as important a part as they do for passenger traffic. Air transportation offers to-day a specialized service for passenger transportation already realized and seeing each year fast improvement in air equipment. Nevertheless, as stated by C. M. Keys, President of the Transcontinental Air Transport Company:[22]

"Only 10% of aviation is in the air; 90% of it is on the ground.

"As regards passenger carrying on a transcontinental scale, that is probably an under-statement; the ratio between ground preparation and actual flight is likely to be wider than that."

For all three classes of performance (mail, express, and passenger), air transportation offers a highly specialized service, superior in speed to any other form. While it is yet seriously handicapped by unfavorable weather conditions, the most unfavorable weather also handicaps highway and railroad service, sometimes for a much longer time.

To build up the general public use of air service, it is already evident that greater attention must be paid to permanence and stability of construction on carefully designed airports, with special attention to the appearance of the airport and the provision of comfort for passengers. The public is already educated to expect and secure comforts and conveniences at railroad stations and bus terminals that were unheard of a few years ago. Without reasonable provision for corresponding conveniences at airports proper development of air transportation cannot be expected to continue steadily at the desired rate. Even in the appearance of the field, the buildings should not be mere sheds or brick barns, but should give, without undue expense, the appearance of careful design and stability suitable for an established industry giving important daily service to the public. Hotels, restaurants, rest rooms, etc., are already making their appearance at American airports as they have for several years past in Europe. Certainly it is reasonable to expect a just proportion of this cost as a proper operating cost of the field to be returned from the field charges for service.

Therefore, in locating the airport, consideration must be given to the ease with which the physical features may be adapted to providing facilities for the comfort and convenience of air passengers. It is also evident that for a number of years to come, the fields will be visited by thousands of people to see shows, races, etc., or merely to observe the daily business of the field. This has already been capitalized at European airports where amusement and athletic parks frequently adjoin the field and where grand stands are frequently supplied for the use of spectators. A voluntary attendance of many thousand people at any municipal air field is an advertising asset far more important than that which can be secured by thousands of dollars worth of paid newspaper and magazine publicity. The impression carried away by these people and passed on by them to their friends will be of the highest importance.

A location for the airport that makes it easily accessible to visitors, a site large enough to provide ample parking space in addition to the bare necessities for planes, and, if practicable, a location and surroundings that present an attractive appearance, are features of real value to the community. Provision of rest rooms and observation stands will encourage the public support of the airport. These are facilities freely provided in any park or other place of public gathering. The airport, to-day, is in many respects more important than the public park, or it should be made so. Paved sidewalks and paved drives and aprons around the hangars are equally important as effecting the comfort of visitors and patrons and keeping them free from mud and dirt.

---

[22] *Chicago Herald and Examiner,* Interview, November 4, 1928.

It has been suggested that present condition of the ground surface will affect the choice of an airport location. This is especially true in so far as it affects the drainage required, etc. It is certain that paved runways for landing and take-off will shortly be a necessity at most of the important airports. A turf or light surface satisfactory a few years ago for the use of light planes will no longer serve the hundreds of heavy transport planes coming into use.

A tabulation has been published[23] which shows twenty-seven standard models of airplanes weighing more than 5 000 lb. A considerable number in transport service to-day weigh more than 10 000 lb. and the Boeing Air Transport Company and the Western Air Express are placing planes in long-distance transport service weighing, respectively, 17 500 and 22 000 lb.

The U. S. Department of Commerce, as a result of tests, states that the landing load should be increased 100% to allow for impact. This brings a wheel load on the surface of the field ranging from 5 000 to 22 500 lb.

Another factor requiring paved runways for safety and economy is the reduced distance required for take-off. On its flight to America, with complete load of fuel, the *Bremen* required a starting run of only 1 800 ft. on the Portland cement concrete runways at Dessau, Germany. At Baldonell, in Ireland, where the Atlantic flight was actually started, 4 750 ft. were required for take-off from the runways of rolled cinders and ashes in spite of the smaller load on the ship.

Despite the shortage of money for building purposes in Germany, concrete runways have been constructed during the three years since 1927 at Dessau, at the Halle-Leipzig Airport, at the Travemünde Airport, Frankfurt-am-Main, and at the Giessen Airport. In this country, concrete runways similar to the highest type of highways have been constructed at the New York City and Detroit Municipal Fields, at the Curtiss-Steinberg Field, at East St. Louis, Ill., and a number of others. It is evident that the character of the present field surface no longer plays as important a part as formerly in determining airport location since for any important airport a paved surface will be provided to support the heaviest planes, reduce the length of take-off, and greatly reduce the maintenance required on the remainder of the field.

Clyde M. Wood,[24] M. Am. Soc. C. E. (by letter).—The factor of an adequate water supply for fire fighting needs more consideration than has been given in the location of most airports. Given two or more locations where the other factors are evenly balanced, the site offering an established, adequate water supply should be selected. The water supply at available sites is generally woefully inadequate or non-existent, but when the reverse is true, advantage should be taken of the fact.

---

[23] *Aviation*, January 18, 1930.

[24] Engr., Executive Div., "Automatic" Sprinkler Corporation of America, Cleveland, Ohio.

When an industrial plant is built with the same degree of isolation, the same number of square feet of floor area, and the same value of building and contents per square foot as is found at the average airport, great care is taken to provide an adequate water supply for protection against fire; and the fire hazard is infinitely more severe in a hangar than in the average industrial building.

The severity of this hazard requires large quantities of water instantly applied not only through the overhead open sprinklers on top of the planes, but also through open floor sprinklers on the under surfaces of wings and fuselage. Therefore, relatively larger supplies are required for hangars. The usual 6-in. or 8-in. main with fair pressure which is sufficient for the average industrial plant is not enough, especially for the larger hangars. For fire protection purposes the water main may be required to supply as much as 3 000 to 4 000 gal. per min., in a hangar, 120 by 120 ft.; and more in larger hangars.

The Detroit City Airport was fortunate in being placed alongside a 36-in. city main with a dependable pressure. On account of the unusual size of this hangar, a 24-in. main was connected to the 36-in. city main, and extended into the hangar. An 8-in. main, because of its close connection to 12-in. and 16-in. mains, was sufficient for the Colonial Hangar at the Newark, N. J., Metropolitan Airport. It is readily seen that because of the friction losses induced by these unusual discharges larger supply pipes are required than are needed for ordinary buildings.

The fact that a gravity tank must be located at a distance from the landing field equal to seven times its height is another reason for selecting a site near an established water supply. Space outside the airport for a tank can be purchased; also right of way for an underground main from tank to hangar, but these considerations are too often overlooked until after the airport site has been selected.

In lieu of a gravity tank as a fire protection supply, large automatic pumps may be used. Here, again, arises the problem of space for the huge reservoir necessary to supply the pumps, for the usual 100 000 gal. is not enough. Somewhere, some day, some engineer is going to provide or utilize a basin beside a land airport for seaplanes, and use the basin water as a reservoir for fire pumps.

That automatic sprinkler protection against fire is one of the major considerations in selecting airport sites is amply attested by the fire record in airplane hangars. The National Fire Protection Association reports a list of fifty-one fires between January, 1928, and July, 1930. The extent of the loss is given for forty-four of these fires as $2 540 700, or an average of $57 743 per fire. Thirty-nine of them destroyed 223 airplanes besides many hangars and much equipment. This is rather "bad medicine" for an infant industry, and a challenge to airport engineers. No other industry is tolerating such fire losses.

Donald M. Baker, M. Am. Soc. C. E. (by letter).—The effort was made in the preparation of this paper to emphasize the relationship of an airport to the entire city or regional plan, and also to set up certain fundamental requirements as a guide for the use of those who are responsible for the selection of sites for airports. The system of rating was suggested primarily as a mechanical device whereby various sites which might be offered for consideration could be compared, one with another. Whenever this method is used as a basis of comparison, the actual items specified are not so important (provided some judgment is used in selecting them), as the method whereby they are applied and the fact that they must be applied uniformly to all subjects being judged. Without question, each of the ten points suggested by the writer could have been subdivided into from two or three to a dozen sub-headings, each of which in itself could have been given a proper weight. This system was used by the writer early in 1928.

The paper referred to by Mr. Fellows[25] illustrates the detail to which such systems of rating may be carried. It was not felt necessary to go into such detail in the writer's paper, as it was thought that any one using the method could, with better results, expand the ten points given. The weights suggested for each factor were suggestive only and might be varied, depending on the different uses to which the field to be selected was put, the main point being that all sites should be compared by the same measuring stick.

The writer does not agree with Mr. Nolen's suggestion as to the elimination of the factor of legislative control. Unless all sites under consideration are within the limits of an incorporated city where zoning and police regulation exist, it is very essential that some measure of control be placed over the development surrounding the airport, in order that height of buildings, use of property, development of highways, etc., may be regulated. If the site lies within unincorporated territory, proper co-operation on the part of county officials may affect this, but if this co-operation cannot be obtained it constitutes a serious drawback.

Accessibility, which should be expressed in terms of time, and not in terms of distance, is primarily an economic factor that can be evaluated in most cases in terms of dollars and cents. In a large metropolitan center, secondary airports will become absolutely essential for private planes used primarily in short hops. These secondary sites, however, can be made relatively small in size, but should be located, of course, so as to be accessible to the better class residential sections of the city wherein will reside most of those using this type of air transportation. The writer is still convinced, however, that the ultimate commercial load in air travel will be the long haul, in which case accessibility to the heart of the city, if maintained within reasonable limits, is not so important. Shore-haul passenger traffic has, of course, been developed to a considerable degree, but until night flying comes into existence as a regular feature, passenger travel between cities 400 to 600 miles apart, will not show much saving of time during business hours. At present, traveling can be done in such cases by rail by leaving the office after closing hours, arriving at one's destination the next morning, having a full business day, and returning the following night, whereas travel by air made during daylight hours does not show any comparable saving of business hours. The gradual reduction in fares for aerial transportation, however, is increasing its popularity. The distance which a major traffic air terminal is located from the heart of the city in the final analysis becomes a question of adjusting capitalized cost of added distance from the business center against the cost of land necessary for the site, and arriving at a minimum total cost. Land cost varies inversely with the distance from the center of the community, and 5 min. in time distance may mean a vast sum in land investment.

---

[25] See footnote, p. 308.

Rapid transit lines are likely to serve the transportation needs of an airport better than highways, although any site should be located so that large crowds may have ready access to it on important occasions. The question of space necessary for landing and take-off areas is one that is still open to discussion. In 1928 much discussion was extant as to the probability of reduced landing and take-off speeds and consequent size of landing and take-off areas, but nothing has appeared during the intervening two years which would indicate that any smaller areas can be used satisfactorily. Ships are getting larger, and more of them are coming into use, resulting in greater crowding of airports. This point also must be kept in mind—that if in the future excess land has been acquired for an airport site, it can always be disposed of at a profit or used to good advantage for park or recreational purposes. The basis of rating given was primarily to apply to the sites to be used and not to the finished field.

The writer agrees with Mr. Angell in his statement as to the desirability of having surface fields, and also with Mr. Marsh as to the need for recreational facilities. This is one situation which can be far better cared for at publicly owned than at privately owned airports. In the latter every possible acre of ground must be made to pay revenue, and a private owner cannot afford to devote a large space to recreational facilities or allow excessively large areas for automobile parking.

The writer heartily agrees with Mr. Nolen's statement that an airport is essentially an industrial establishment, not only through the attendant activities and development which will surround it, but by virtue of the noise, crowds, dust, etc., which must exist even at the best regulated port.

Mr. Grimm's discussion and the re-arrangement of governing factors brings out a point the writer had in mind. The factors must be given different weights when applied to fields used for different purposes. Accessibility and transportation are far more important in a small field used by owners of private planes for business or pleasure purposes than in the case of a major terminal, and, at the former, meteorology is not so important since departures and landings are not usually made on schedule. The factors given by Mr. St. John illustrate the same thing, although the writer believes that probably too much weight is given by him to meteorology, which totals 30 points in Factors 1, 2, and 6. It is not thought that Item No. 13 in Table 1 is necessary. The airplane factory sites should come to the airport and the port should not go to the factory. His Item No. 7, "Prospective Neighborhood Development", would be one sub-factor under the writer's factor, "Legislative Control."

The writer is very familiar with the excellent work being done by the Los Angeles County Regional Planning Commission in the development of a regional plan of airports in that county, which is referred to by Mr. Fox. The latter's discussion brings out very admirably the method and study which was suggested by the writer as a preliminary to the selection of any site or sites, and emphasizes the necessity of a comprehensive airport plan for any region.

Aeronautical transportation is here to stay, and the sooner the fact is accepted by the general public, including officials, the sooner they will realize that a regional plan of airports is just as essential as a regional plan of highways, and the more money will be saved in the ultimate acquisition of such sites. Mr. Fox's illustration of earning ability of Air Terminal No. 8, in the San Gabriel Valley, California, is very apt. It shows the proper economic approach to the matter and develops the fact that in the future airports can undoubtedly be classed as publicly owned utilities of a self-supporting nature.

The writer is much gratified with the general discussion which the paper has brought out, and hopes that the importance of airports in the general city or regional plan has been emphasized. Private companies, as well as public agencies, should realize that the location of an airport is a major factor making for its success, both from the standpoint of the port itself and of its service to the community. Much has been learned by railroads through experience in improper location of terminals and subsequent re-arrangements. No site should be chosen by any community until its location has been given the most thorough study.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

TRANSACTIONS

Paper No. 1770

SPILLWAY DISCHARGE CAPACITY OF WILSON DAM[1]

BY LOUIS G. PULS,[1a] ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. CECIL E. PEARCE, J. H. JONES, AND LOUIS G. PULS

SYNOPSIS

This paper describes the current-meter discharge measurements made on the spillways at Wilson Dam. The readings were taken for several gate-openings from 3 ft. to full gate and under two conditions, namely, no discharge and full discharge through the adjacent spillways. The head on the crest was 18 ft. Discharge quantities, together with the calculated coefficients for the several partly opened gates and also for full gate, have been obtained.

INTRODUCTION

In view of the fact that nearly every dam built recently has an overflow spillway, the discharge of which is controlled by regulating gates of various types, there is a great need for information as to the discharge capacity of these openings. Although experiments have been made and results published on sharp-edged weirs discharging under low heads, very little has been done to supply data as to discharge capacities of spillways with round crests and variable gate-openings under the higher heads, for example, those up to 18.0 ft. It is not a surprising circumstance that such investigations have not been made. Entirely aside from the expense involved, it is not an easy matter to control conditions in order to obtain this information accurately, because a jet of water, 15 ft. in depth, traveling at the rate of 27 ft. per sec., is a rather treacherous mass of energy.

Nevertheless, this work was attempted, inspired by its need and value to hydraulic science, and results have been obtained which are here presented as a pioneering effort.

DESCRIPTION OF SPILLWAY

The spillway section consists of a gravity overflow dam with the crest about 80 ft. above the bed of the river. On the crest there are provided fifty-eight spillway openings formed by the spacing of concrete piers which serve the double purpose of supporting the regulating gates and the arches of a highway bridge. The clear openings are each 38 ft., and the piers are 8 ft. thick. The gates are 39 ft. 3 in. long by 18 ft. high.

---

[1] Published in October, 1929, *Proceedings*.

[1a] Care, Hugh L. Cooper & Co., Inc., Kitchkas, Ukraine, Union of Socialistic Soviet Republics.

The profile of the top of the overflow section, up stream from the seal, has the shape of an ellipse, with the equation, $\frac{x^2}{36} + \frac{y^2}{9} = 1$, the major axis being horizontal and 3 ft. below the crest. (See Fig. 1 (*a*).) The horizontal distance from the up-stream face of the dam to the gate seal is 6 ft. Down stream from the seal the curve is a parabola, with the equation, $42.585\ x = y^2$, the axis of which is vertical and coincides with the minor axis of the ellipse, the origin being at the crest. The up-stream side of the piers is semi-circular in horizontal section, with a radius of 4 ft., that forms a half round nose against the flow of water.

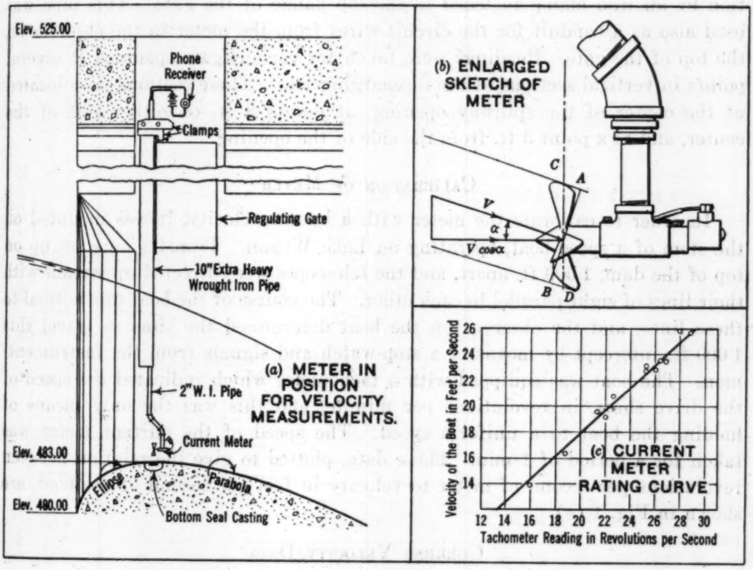

Elev. 525.00
Phone Receiver
Clamps
Regulating Gate
10" Extra Heavy Wrought Iron Pipe
2" W. I. Pipe
Current Meter
Elev. 483.00
Elev. 480.00
Ellipse
Parabola
Bottom Seal Casting
(a) METER IN POSITION FOR VELOCITY MEASUREMENTS.
(b) ENLARGED SKETCH OF METER
C
A
V
α
V cosα
B
D
Velocity of the Boat in Feet per Second
26
24
22
20
18
16
14
12
14
16
18
20
22
24
26
28
30
Tachometer Reading in Revolutions per Second
(c) CURRENT METER RATING CURVE

FIG. 1.—ARRANGEMENT OF MEASURING DEVICES.

METHOD OF MEASUREMENT

The Price meters available were designed primarily for comparatively low velocities, up to 4 or 5 ft. per sec. Therefore, it was necessary to construct a meter with bearings capable of withstanding the high velocities of the spillway flow, which are as much as 27 ft. per sec. This meter was designed and built by the writer in the job machine shops. The propeller has three blades, is 4 in. in diameter, and is mounted on a small shaft which has thrust-bearing against the supporting casting. A worm was machined on the end of the shaft, so that it meshed with a wheel, into the face of which was set a small pin.

This pin, in revolving, came in contact with a fine wire terminal and thereby closed a dry-cell battery circuit and caused a clicking sound in the headphone. The ratio of the speed of the propeller to the worm wheel was 20 to 1.

This meter was mounted on the end of a 2-in. wrought-iron pipe in such a manner that the plane of the propeller coincided with the plane of the up-stream side of the gate (see Fig. 1 (*a*)). For all readings at points deeper than 4 ft. in flowing water, a 10-in., extra heavy, wrought-iron pipe supported the 2-in. pipe. In order to place the propeller in the plane over the seal, it was necessary to offset the meter a few inches down stream, and this was done by 1-in. pipe-and-ell fittings connected to the end of the 2-in. pipe (See Fig. 1 (*b*).)

The 2-in. pipe support was sufficiently strong to allow the meter to penetrate 4½ ft. of flowing water. This support was about 32 ft. long and, when placed vertically against the up-stream side of the gate, was held in that position by an iron clamp fastened to the top flange of the gate. This pipe was used also as a conduit for the circuit wires from the meter to the observer on the top of the gate. Readings were taken for various gate-openings at several points in vertical sections of the discharging jet. These sections were located at the center of the spillway opening, at points 8 ft. on either side of the center, and at a point 3 ft. from the side of the opening.

Calibration of Meter

In order to calibrate the meter with a known velocity, it was mounted on the stem of a speed-boat, operating on Lake Wilson. Transits were set up on top of the dam, 1 000 ft. apart, and the telescopes were directed up stream with their lines of sight parallel to each other. The course of the boat was normal to these lines, and the observer on the boat determined the time to travel this 1 000-ft. intercept by means of a stop-watch and signals from the instrumentman. The boat was equipped with a tachometer which indicated the speed of the drive shaft, in revolutions per minute, and this was the only means of holding the boat to a uniform speed. The speed of the current meter was taken for a period of 1 min. These data, plotted to give the relation between revolutions per second of meter to velocity in feet per second of the boat, are shown in Fig. 1 (*c*).

Current Velocity Data

As is customary, the metered results obtained in the field were arranged in tabular form. The gate-opening for each reading was given; and the elevation of pool level, the exact position of the meter in the discharging area, and the speed of the propeller were likewise given. From the calibration curve (Fig. 1 (*c*)), the velocity of the water was computed and, finally, this velocity was corrected to give results under a uniform head of 18.0 ft. which placed the pool level at Elevation 501.0 (normal operating pool level).

In these measurements it has been assumed that the velocity as recorded by the meter is the horizontal component of any diagonal flows which may exist. The shaft of the meter was supported as rigidly as possible in a horizontal position. Thus, in Fig. 1 (*b*), it is most reasonable to assume that the velocity,

$V \cos \alpha$, multiplied by the area in the plane, $C D$, is equal to the velocity, V, multiplied by the area in the plane, $A B$. The area in the plane, $C D$, is in the vertical plane over the crest.

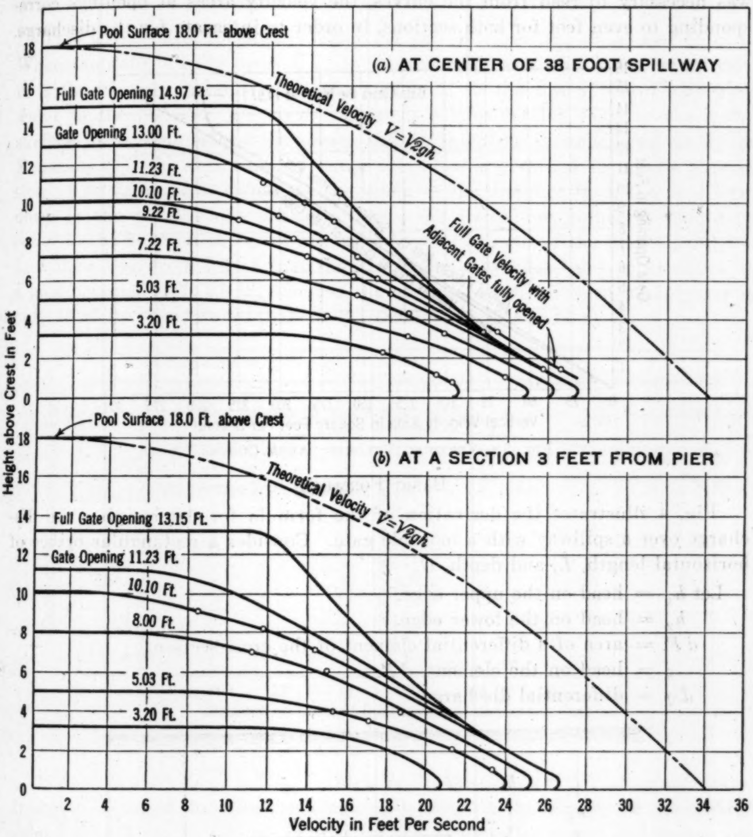


FIG. 2.—VERTICAL VELOCITY CURVES.

From the data thus collected curves were drawn showing the relation between gate-opening and the intensity of velocity for various depths below the bottom of the gate. Fig. 2 (*a*) shows the vertical velocity curves for a section at the center of the gate, and Fig. 2 (*b*), those for a section 3 ft. from one end of the spillway.

The areas under the curves of Fig. 2 (*a*) and Fig. 2 (*b*) were measured with a planimeter, and the results obtained were plotted so as to give a curved-line relation between the gate-opening, in feet, and the vertical velocity area, in square feet per second, for the two sections of the discharging jet; that is, in Fig. 3, the values plotted for the vertical velocity areas are the products of water depth and average velocity for the various gate-openings shown in Fig. 2 (*a*) and Fig. 2 (*b*). Since these points were at odd gate-openings, it was necessary to read from the curves, the velocity areas at openings corresponding to even feet for both sections, in order to integrate for the discharge.

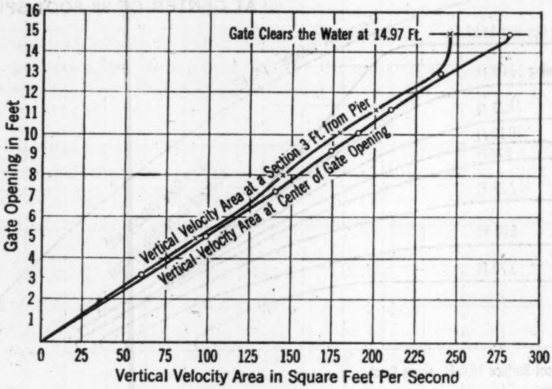


FIG. 3.—VERTICAL VELOCITY AREA CURVES.

BASIC FORMULA

Fig. 4 illustrates the derivation of the formula for determining the discharge over a spillway with a movable gate. Consider a rectangular orifice of horizontal length, L, and depth, G.

Let h_1 = head on the upper edge.
h_2 = head on the lower edge.
dF = area of a differential element of the cross-section.
z = head on the element, dF.
dq = differential discharge.

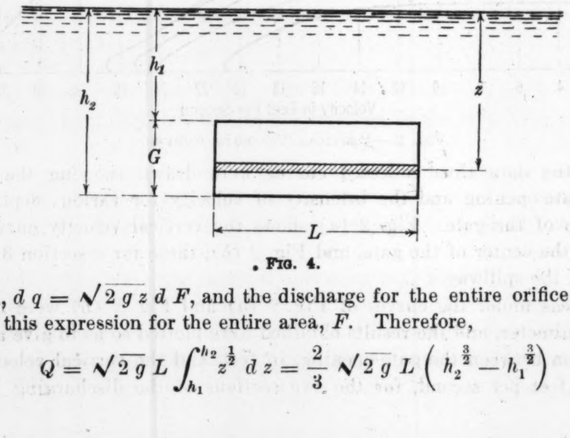


FIG. 4.

Then, $dq = \sqrt{2\,g\,z}\;dF$, and the discharge for the entire orifice is the integral of this expression for the entire area, F. Therefore,

$$Q = \sqrt{2\,g}\;L\int_{h_1}^{h_2} z^{\frac{1}{2}}\;dz = \frac{2}{3}\sqrt{2\,g}\;L\left(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}\right)$$

For the actual case, the rate of discharge is found by applying a coefficient to this ideal value. The result may be written,

$$Q = m\,L\left(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}\right)\dots\dots\dots\dots\dots\dots(1)$$

This is the basic formula. When the gate is closed, $h_1 = h_2$ and $Q = 0$. When the gate is fully opened, h_1 is theoretically zero, but in an actual case it is the vertical distance from the pool level to the top of the discharging water in the plane of the crest. There is a considerable slope to the water surface as it approaches the crest, and if this fall is considered as h_1 in Equation (1), the full gate discharge can also be obtained from this expression. The value of h_1 should not be taken as the distance from the pool level down to the point where the gate clears the water, but rather it should be equal to the head on the crest minus the average depth of the discharging jet. It will be seen from Fig. 5 that the gate completely clears the upper nappe at a gate-opening of 14.97 ft., whereas the average depth of overflowing water is only 13.95 ft. This would make $h_1 = 18.00 - 13.95 = 4.05$ ft.

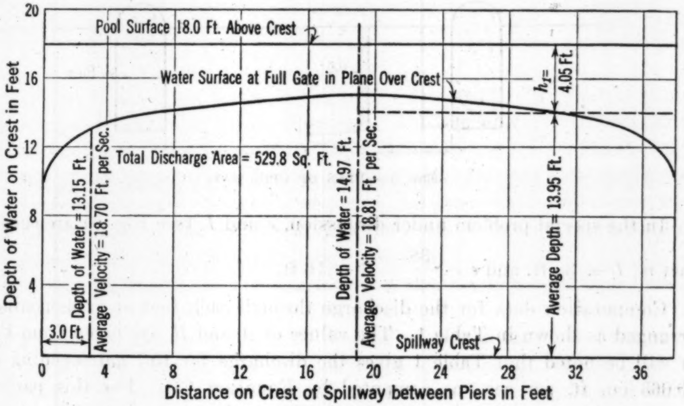


FIG. 5.—WATER SURFACE AT THE CREST WITH FULL GATE-OPENING.

If $h_1 = 0$, then $Q = c\,L\,h_2^{\frac{3}{2}}$, which is the formula for a rectangular weir. It should be noted that the coefficient, *m*, is used, in this study, for the rectangular orifice formula, while *c* is used for the rectangular weir. The reason for this is that the coefficient for the latter is different, due to the fact that h_1 is assumed equal to zero.

DISCHARGE FORMULA WITH GATE PARTLY OPENED

To determine discharge it is necessary to express the vertical velocity area as a function of the length of the spillway and to integrate between the limits of this length. Since it varies increasingly from the pier to the center of the gate-opening, according to some function (most probably of the second degree), the vertical velocity area can be expressed as a function of this distance by the equation of the parabola. No doubt there is a rapid decrease in the rate of change in this area within a few inches from the masonry pier and, consequently, the curve departs from any definite function of the spillway length. However, the percentage of error is small and can be neglected. The equation of the parabola:

$$x^2 = 4\,p\,(y - A) \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

is established by solving for the constant, p, in terms of A, B, and e (see Fig. 6). The vertical velocity area, as found from Fig. 3, is here designated as the ordinate, A, and in moving according to Equation (2), between the limits of $x = 0$ and $x = \frac{L}{2}$, it will generate a volume representing one-half the discharge. The derived equation for the total discharge of one spillway will be,

$$Q - A\,L = \frac{(A - B)\,L^3}{12\,e^2} \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

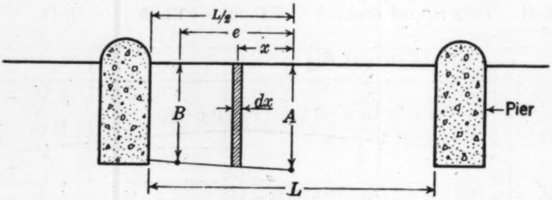


FIG. 6.—PLAN OF SPILLWAY.

In the special problem under discussion, e and L (see Fig. 6) are constant; that is, $L = 38$ ft. and $e = \frac{38}{2} - 3 = 16$ ft.

Computation data for the discharge through each foot of gate-opening are arranged as shown in Table 1. The values of A and B, are taken from Fig. 3. It will be noted that Table 1 gives the discharge for full gate-opening to be 10 063 cu. ft. per sec., as computed by Equation (3). For this particular opening, the parabolic curve integration may give a result greater than the actual, due to the greater difference in the velocity areas, A and B. If the velocity area at A is divided by the depth of the water at that point (see Fig. 5), the average velocity for the section is $\frac{281.6}{14.97} = 18.8$ ft. per sec. Similarly, the average velocity for the section, B, is $\frac{245.9}{13.15} = 18.7$ ft. per sec. This shows the average velocity to be nearly equal in the two sections. The total area of the discharging jet at full gate is 529.8 sq. ft. (see Fig. 5). This will give a discharge of $529.8 \times 18.75 = 9\,925$ cu. ft. per sec.

The surface of the water is not level from pier to pier in the plane of the crest at full gate-opening. The round-nosed piers do not fully suppress the contraction at the surface, the water being compelled to make further contraction because of the diagonal flows. This crowding tendency toward the center forms the elliptical curve as shown in Fig. 5. It is noted that the gate would clear the water at the 14.97-ft. opening, while the average depth is 13.95 ft. In computing values for the coefficient, *m*, in Fig. 7, this average depth was subtracted from the total head of 18.0 ft. on the crest to obtain the value of h_1, which is 4.05 ft. This is why it has been correlated to "Average Depth of Discharging Jet" instead of "Gate-Opening". The coefficient, *m*, was computed for these conditions by Equation (1). The points were then plotted as shown in Fig. 7 and a smooth curve was drawn through them. Revised ideal coefficients were then read from the smooth curve and the final discharge values were computed from them. The coefficient, *c*, in the Francis formula, corresponding to this coefficient, *m*, is 3.420 for full-gate discharge.

TABLE 1.—COMPUTED DISCHARGE WITH ADJACENT GATES CLOSED.

Gate-opening, in feet.	*A.*	*B.*	*Q*, in cubic feet.
1	19.2	19.0	726
2	39.3	38.2	1 474
3	58.6	57.2	2 202
4	78.3	77.0	2 952
5	98.2	96.0	3 692
6	117.6	114.0	4 404
7	136.2	130.6	5 076
8	153.5	145.6	5 692
9	170.8	160.6	6 308
10	188.6	178.0	6 978
11	205.5	197.0	7 657
12	224.0	216.5	8 378
13	243.6	236.5	9 130
Full gate	281.6	245.9	10 063
Corrected value	.....	.....	9 925

The discharge curves on Fig. 7 are plotted against gate-openings, thereby giving the maximum discharge at the instant the gate clears the water. As the gate rises through the upper 12 in. of flowing water, the tangent of the curve rapidly approaches infinity, and the increment of discharge becomes less for any increment of gate-opening until the gate clears the water, beyond which instant, obviously, the increment of discharge is zero.

DISCHARGE CAPACITY OF SPILLWAY WITH ADJACENT GATES OPEN

The discharge of one spillway is increased materially if the adjacent openings are discharging at the same time. This increase in quantity discharged can be treated as if it were due to an increment of head. Thus, if the head, *H*, produces a discharge, *Q*, with adjacent gates closed, $H + \Delta h$ will produce $Q + \Delta q$ with them opened, wherein *Q* can have, without appreciable error, the same ratio to $Q + \Delta q$ as the ratio of the three halves power of the heads, or,

$$\frac{Q}{Q + \Delta q} = \left(\frac{H \Delta}{H + \Delta h}\right)^{\frac{3}{2}}$$

Since the values of *H* and *Q* are known (see Table 1), it remains to find $H + \Delta h$ before it is possible to solve for $Q + \Delta q$. In order to determine the head, $H + \Delta h$, it is only necessary to meter the velocity at a short distance above the crest, and then solve for the increment of head necessary to increase the velocity curve, under the condition with adjacent gates closed, to coincide with the metered points under the condition with the adjacent gates opened.

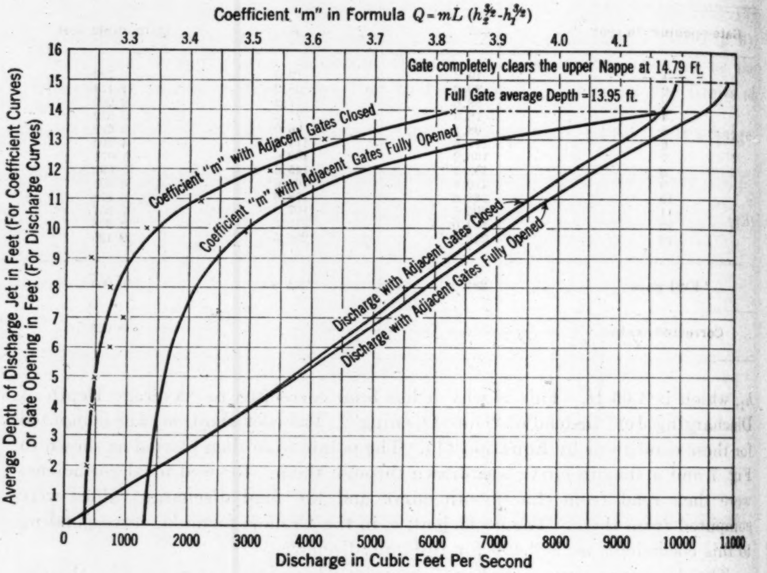


FIG. 7.—COEFFICIENT AND DISCHARGE CURVES.

At the center of the spillway, with full gate-opening, at an elevation of 2.0 ft., above the crest, the velocity is 24.8 ft. per sec. under the first condition and 25.6 ft. per sec. under the second condition (see Fig. 2 (*a*)). Since the head of 16.0 ft. produced the velocity of 24.8 ft. per sec., a head of $16\left(\frac{25.6}{24.8}\right)^2 = 17.08$ ft. may be expected to produce the velocity of 25.6 ft. per sec. This is equivalent to raising the pool level to $17.08 + 2.0 = 19.08$ ft. above the crest instead of 18.0 ft. as in the first condition. It follows that the effect on the discharge of the intermediate spillway of opening the neighboring gates fully, is similar to that of raising the pool 1.08 ft.

By using the formula,

$$Q' = Q\,\frac{h_4^{\frac{3}{2}} - h_3^{\frac{3}{2}}}{h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}} \quad \text{.........................} (4)$$

in which, Q' equals $Q + \Delta q$, and h_4 equals $H + \Delta h$, the discharge, Q', can be found because all other values are now known. Equation (4) holds good not only for full gate-opening, but also for gates partly opened, in which the values of h_3 and h_1, the respective heads on the top of the jet, must be introduced.

The curves of Equation (4) are plotted in Fig. 7 for the two conditions: Adjacent gates closed and adjacent gates fully opened. By picking the proper discharge values from these curves, the corresponding values of m may be determined by making the necessary substitutions in Equation (1). The corresponding, deduced coefficient, c, in the Francis formula for full gate discharge, is 3.738.

Discharge of One Opening Under Condition of Adjacent Gates Partly Opened

Fig. 7 shows the discharge of one opening under the conditions that both adjacent gates are either closed or fully opened. Although no measurements were made when one adjacent gate was open and the other closed, it is reasonable to assume that, under this condition, the discharge would be the mean of those determined from the two curves. Thus, if one adjacent gate were closed and the other fully open, the discharge of the intermediate 8.0-ft. opening would be $\frac{5\,583 + 5\,801}{2} = 5\,692$ cu. ft. per sec.

If the adjacent gates were open a small distance compared to the intermediate gate, the effect on the discharge of the latter, no doubt, would be negligible.

If each of the adjacent gates was opened as much as the intermediate gate, the effect would be slightly less than under the condition of adjacent gates fully opened, because every particle of water in the neighboring regions for the depth of the opening is in motion and, therefore, the retarding influence is removed.

Total Spillway Capacity of Wilson Dam

With adjacent gates fully opened, the discharge of one 38-ft. spillway under a head of 18 ft. on the crest at full gate, is 10 848 cu. ft. per sec. This would give a total spillway capacity of 10 848 × 58 (number of 38-ft. openings) = 629 184 cu. ft. per sec. Furthermore, if two or more gates were open on both sides of any discharging opening, this condition would increase the capacity of the latter slightly. There is a limit no doubt to this increased discharge effect by opening more gates. Perhaps the effect would not be appreciable beyond two, but, regarding this limit, no information on which calculations can be based, is available.

Comparison Between Florence Gauge Rating and Wilson Dam Spillway Gate-Opening Rating

The United States Geological Survey gauge at the Florence (Ala.) Bridge is located about 3 miles down stream from the Wilson Dam. This offers an opportunity to make comparative calculations of the Tennessee River discharge. The Florence gauge was established in 1871 and a rating curve has been used for it since 1894. The control at the section is considered practically permanent. On this rating curve appear approximately 120 points indicating current-meter measurements taken from a range of 8 000 to 319 000 sec-ft.

For the purpose of comparison, data were taken from the records of the plant at Wilson Dam over a period such that the discharge ranged from 19 000 to 345 000 cu. ft. per sec., using twenty-four different flows within this range. These data included: (1) The setting of each of the fifty-eight spillway gates; (2) the trash chute opening; and (3) the quantity of water passing through the turbines based upon power-discharge curves. From these data, the total discharge of the river was determined, using the foregoing rating for the spillways (see Fig. 7). The Florence gauge discharges were taken from the official river records for the exact time when the recorded flows at Wilson Dam reached the gauge. Points were selected only when the gauge showed a constant reading for six hours. This allowed ample time for travel between the stations.

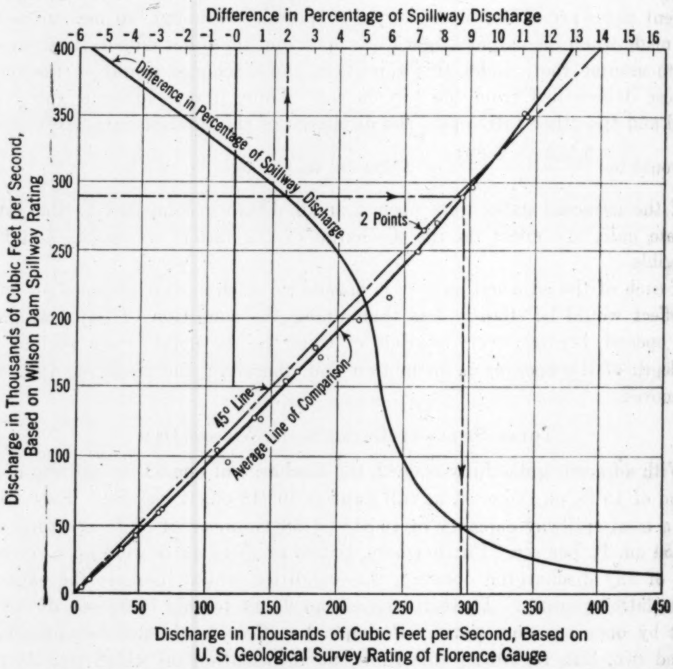


FIG. 8.—DISCHARGE OF TENNESSEE RIVER AT WILSON DAM BASED ON SPILLWAY RATING COMPARED WITH DISCHARGE AT FLORENCE GAUGE BASED ON U. S. GEOLOGICAL SURVEY RATING.

The data for the spillway computations are shown in Table 2. These discharges were listed in Table 3 and were added to the discharges through the turbines and trash chute, thereby giving the total flow past the dam. The official gauge heights and discharge rating of the Florence gauge are also included in Table 3 for purposes of comparison.

It will be noted that in all but the last two instances the discharge as shown by the Florence gauge rating is greater than that by the spillway

TABLE 2.—DISCHARGE MEASUREMENTS OF TENNESSEE RIVER AT WILSON DAM.

Date.	Time.	Head on crest, in feet.	Gate-opening, in feet.	Number of gates open.	Discharge for one gate, in cubic feet per second.	Discharge for total number of gates, in cubic feet per second.	Total discharge of spillway, in cubic feet per second.
September 16, 1926.....	10:00 P. M.	17.33	0.5	11	390	4 290	4 290
September 10, 1926.....	4:00 P. M.	17.49	0.5	4	392	1 568	
			1.0	11	784	8 624	10 192
August 24, 1926.........	11:00 P. M.	17.40	3.0	3	2 281	6 843	
			4.0	12	2 997	35 964	42 807
December 10, 1926......	8:00 A. M.	17.94	2.5	4	1 941	7 764	
			3.0	7	2 317	16 219	
			3.5	2	2 682	5 364	
			4.0	1	3 046	3 046	32 393
February 2, 1927........	2:00 A. M.	17.11	2.0	2	1 528	3 056	
			6.5	6	4 650	27 900	
			7.0	6	4 974	29 844	60 800
February 8, 1927........	8:00 P. M.	17.10	1.0	1	774	744	
			1.5	1	1 151	1 151	
			2.0	1	1 528	1 528	
			6.5	12	4 650	55 800	59 253
February 21, 1927.......	4:00 A. M.	17.05	1.5	1	1 150	1 150	
			2.0	2	1 528	3 056	
			3.0	1	2 258	2 258	
			4.0	9	2 968	26 712	
			8.0	11	5 607	61 677	94 853
February 16, 1927.......	8:00 P. M.	17.11	1.5	1	1 150	1 150	
			2.0	2	1 528	3 056	
			5.5	8	3 992	31 736	
			8.0	12	5 607	67 284	103 226
February 24, 1927.......	10:00 P. M.	17.14	1.5	1	1 150	1 150	
			2.0	2	1 528	3 056	
			3.0	1	2 258	2 258	
			8.0	17	5 607	95 319	
			9.0	4	6 230	24 920	126 703
December 17, 1926......	12:00 Midnight	17.88	1.5	1	1 178	1 178	
			2.0	2	1 566	3 132	
			5.0	6	3 758	22 548	
			6.0	8	4 446	35 568	
			8.0	11	5 778	63 558	125 984
February 28, 1927.......	3:00 A. M.	17.18	1.5	1	1 155	1 155	
			2.0	2	1 533	3 066	
			3.0	1	2 266	2 266	
			7.0	5	4 992	24 960	
			8.0	8	5 629	45 032	
			12.0	10	8 305	83 050	159 529
February 27, 1927.......	8:00 A. M.	17.15	1.5	1	1 155	1 155	
			2.0	2	1 533	3 066	
			3.0	1	2 266	2 266	
			4.0	1	2 977	2 977	
			8.0	12	5 629	67 548	
			11.0	10	7 565	75 650	152 662
March 16, 1927..........	12:00 Noon	17.11	2.0	1	1 528	1 528	
			4.0	16	2 968	47 488	
			8.0	8	5 607	44 856	
			11.0	7	7 531	52 717	
			12.0	3	8 266	24 798	171 387
March 12, 1927..........	10:00 A. M.	17.13	2.0	10	1 528	15 280	
			3.0	1	2 258	2 258	
			5.0	1	3 659	3 659	
			6.0	1	4 325	4 325	
			8.0	11	5 607	61 677	
			13.0	10	9 071	90 710	177 909

TABLE 2.—(*Continued*).

Date.	Time.	Head on crest, in feet.	Gate-opening, in feet.	Number of gates open.	Discharge for one gate, in cubic feet per second.	Discharge for total number of gates, in cubic feet per second.	Total discharge of spillway, in cubic feet per second.
December 25, 1926......	6:00 P. M.	17.94	3.0	1	2 317	2 317	
			4.0	1	3 046	3 046	
			8.0	14	5 778	80 892	
			Full	12	10 757	129 084	215 399
March 15, 1927..........	5:00 P. M.	17.09	2.0	1	1 528	1 528	
			4.0	10	2 968	29 680	
			5.0	6	3 659	21 954	
			8.0	8	5 607	44 856	
			Full	10	10 044	100 440	198 458
January 4, 1927.........	5:00 P. M.	17.65	3.0	3	2 303	6 909	
			4.0	1	3 026	3 026	
			5.0	2	3 734	7 468	
			8.0	9	5 737	51 633	
			Full	19	10 578	200 982	270 018
December 27, 1926......	9:00 A. M.	17.68	3.0	1	2 303	2 303	
			4.0	1	3 026	3 026	
			8.0	10	5 737	57 370	
			Full	19	10 578	200 982	263 681
December 26, 1926......	8:00 A. M.	17.67	3.0	1	2 303	2 303	
			4.0	1	3 026	3 026	
			8.0	11	5 737	63 107	
			Full	17	10 578	179 826	248 262
December 27, 1926......	5:00 P. M.	17.66	3.0	1	2 303	2 303	
			4.0	1	3 026	3 026	
			8.0	10	5 737	57 370	
			Full	19	10 578	200 982	263 681
March 13, 1927..........	2:00 P. M.	17.31	2.0	1	1 538	1 538	
			4.0	14	2 988	41 832	
			6.0	6	4 355	26 130	
			8.0	2	5 651	11 302	
			10.0	3	6 922	20 766	
			Full	19	10 221	194 199	295 767
January 3, 1927.........	5:00 A. M.	17.72	3.0	3	2 303	6 909	
			4.0	1	3 026	3 026	
			5.0	2	3 734	7 468	
			8.0	6	5 737	34 422	
			10.0	2	7 036	14 072	
			Full	21	10 578	222 138	288 085
December 29, 1926......	4.00 P. M.	17.78	4.0	17	3 037	51 629	
			5.0	5	3 747	18 735	
			8.0	4	5 758	23 032	
			10.0	2	7 065	14 130	
			12.0	2	8 537	17 074	
			Full	21	10 668	224 028	348 628
December 30, 1926......	10:00 P. M.	17.68	4.0	11	3 026	33 286	
			5.0	11	3 734	41 074	
			8.0	4	5 737	22 948	
			10.0	2	7 036	14 072	
			12.0	2	8 499	16 998	
			Full	21	10 578	222 138	350 516

rating. The discharges in Columns (2) and (3) of Table 3 were deducted from those in Column (6) and the net quantities were plotted against corresponding values in Column (1), to give the average relation line as shown on Fig. 8. By drawing the 45° line, which would be the locus of all the points if the two methods checked exactly, it will be seen that the actual average relation line is below the 45° line from zero to 320 000 cu. ft. per sec., and there it crosses and remains above it. Calculating the ordinate intercept between these two lines in percentage of the spillway discharge and plotting this percentage to the latter quantity, gives the compound curve marked, "Difference in Percentage of Spillway Discharge", part of which is negative and part positive.

TABLE 3.—DISCHARGE OF TENNESSEE RIVER AT WILSON DAM BASED UPON THE SPILLWAY RATING IN COMPARISON WITH THE DISCHARGE AT FLORENCE GAUGE BASED UPON THE UNITED STATES GEOLOGICAL SURVEY RATING.

Discharge of spillway taken from Table 2, in cubic feet per second. (1)	Discharge of trash chute, in cubic feet per second. (2)	Discharge of turbines, in cubic feet per second. (3)	Total discharge at Wilson Dam, in cubic feet per second. (4)	Florence gauge, in feet. (5)	Discharge on U. S. Geological Survey rating, in cubic feet per second. (6)
4 290	310	12 905	17 505	1.42	19 100
10 192	310	9 777	20 279	1.76	21 200
32 393	890	12 943	46 226	5.31	49 600
42 807	0	4 790	47 597	5.52	51 500
60 800	635	14 796	76 231	8.45	82 000
59 258	635	14 720	74 608	8.32	80 600
94 853	635	108	95 596	11.40	118 000
103 226	635	9 344	113 205	11.40	118 000
126 703	635	574	127 912	12.95	138 500
125 984	890	97	126 971	13.20	141 500
159 529	635	108	160 272	15.25	170 000
152 662	635	97	153 394	14.65	160 400
171 887	1 475	7 195	180 057	17.00	195 900
177 909	635	6 007	184 551	16.62	189 200
215 339	890	137	216 866	19.91	239 600
198 458	1 475	7 192	207 125	18.98	225 000
270 018	890	4 385	275 293	22.60	280 500
263 681	890	1 838	266 409	22.04	268 200
248 262	890	97	249 249	21.34	261 500
263 681	890	447	265 018	21.67	266 000
295 767	635	380	296 782	23.91	308 200
288 035	890	3 487	292 412	23.40	295 400
348 628	890	2 285	351 803	26.51	346 000
350 516	890	561	351 967	26.35	343 100

To illustrate its use in comparing the two sets of data, consider a spillway discharge of 290 000 cu. ft. per sec. A horizontal line from 290 intersects the percentage curve at 2, so that the corresponding discharge, determined by the Florence gauge rating, would be 290 000 + 0.02 × 290 000 = 295 800 cu. ft. per sec. This value may be checked by producing the horizontal line to intersect the average curve of comparison. The example here given excludes consideration of discharges through trash chute or turbines which must naturally be added to the quantity resulting from the use of Fig. 8.

ACKNOWLEDGMENT

This study of spillway discharge was made possible only by the generous interest and co-operation of Capt. Hugh P. Oram, Corps of Engineers, U. S. Army, District Engineer, Florence, Ala., who authorized both the time of the personnel and the use of equipment necessary for making the tests.

DISCUSSION

Cecil E. Pearce,[2] Assoc. M. Am. Soc. C. E. (by letter).—As the author states, there are relatively few data in regard to flows through large spillways as deep as 18 ft. When spillways are designed for such heads, one might, in calculating the discharge of a large rounded ogee spillway section, use a conservative value, say, $c = 3.4$, and hope that the spillway would discharge at least as much as computed by using that coefficient in the formula:

$$Q = c\,L\,H^{\frac{3}{2}} \qquad (5)$$

Therefore, when such measurements as those presented by the author appear, engineers who are interested immediately greet them "with open arms" and analyze them to see if there are data which may be of use to them.

The writer does not agree with the author in his mathematical treatment of "Discharge Capacity of Spillway with Adjacent Gates Open." Mr. Puls has stated that "this increase in quantity discharged can be treated as if it were due to an increment of head."

It seems more logical to the writer to attribute the increase in discharge due to adjacent gates being open as caused by a lessening of the end contractions, because it can easily be seen that with only one gate open, the water would flow diagonally into the spillway opening, as well as straight into it. This would tend to cause more end contraction than when the adjacent gates are open and the water is flowing more nearly straight through the several spillway openings. If that is correct reasoning, the relation between Q (one gate open) and Q' (adjacent gates open) depends upon formulas having to do with end contractions. Regarding end contractions, Hoskins[3] states:

"Francis concluded that each contraction decreases the effective length of the weir by an amount proportional to H, and gave the following formula:

$$Q = 3.33\,(b - 0.1\,n\,H)\,H^{\frac{3}{2}}$$

in which (n) is the number of end contractions, usually two."

Other authors such as King[4] and Gibson[5] also cover the same subject, stating in effect that nothing better has yet been found, and that the formula seems to be rational.

Quotations from these and other authorities will not be made herein, but are mentioned for the purpose of showing that, from the best information available on the subject, the discharge does not change in proportion to the three-halves power on any value of $\left(\frac{H}{H + \Delta h}\right)$, but end contractions diminish the discharge from Q (without end contractions) to an amount equal to $\left(\frac{L - n\,k\,H}{L}\right) Q$.

---

[2] Chf. Designing Engr., Water Dept., Pasadena, Calif.

[3] "Text Book on Hydraulics", L. M. Hoskins, Third Edition, 1911, p. 149.

[4] "Handbook of Hydraulics", Horace W. King, First Edition, p. 67.

[5] "Hydraulics and Its Applications", A. H. Gibson, Second Edition, pp. 141–142.

It is thus seen that the change in discharge is not even proportional to H, but for any given values of H and L, it is a constant with L in the denominator, such that if L is large enough, the change occasioned by end contractions is small in proportion to the total. For these reasons, the writer believes that Equation (4) does not correctly show the relation which exists between Q and Q'.

The author also states that "the discharge, Q', can be found because all other values are now known." Even if Equation (4) were correct, the absence of any mention of any measured value of h_3 leads one to believe that the author has assumed that $h_3 = h_1$ and calculated Q' accordingly from Equation (4), obtaining 10 848 sec-ft. The writer believes that it is not necessarily correct to assume $h_3 = h_1$ in the case of gate fully open, and, therefore, h_3 and Q' are both unknown; so that for any correct solution based on experiment, the value of h_3 should also have been measured. However, even if the value of h_3 had been measured, it is believed that Equation (4) is irrational and could not be expected to give rational results.

The derivation of Equation (4) implies that Q' is to be calculated from it; otherwise, it has no use. Hence, it is assumed that Q' was calculated and not obtained from actual measurement of velocities over the entire section, and that, therefore, the value, $Q' = 10\,848$, cannot be used as a check on Equation (4).

If the author's presentation is correctly understood, the writer believes that insufficient data are presented for any reasonably accurate calculation of Q' (gate fully open) which could purport to depend on the field observations; or, at least, if sufficient data do exist, the mathematical presentation given by the author does not solve the problem.

The author's determination of $c = 3.420$ for one gate open, however, is as nearly correct as the data warrant, and for conditions similar to those at the time of measurement at Wilson Dam, the value of c found ought to be fairly reliable.

The writer believes that the velocity of approach should receive more attention, in general, by all investigators, in order to make the results of observation and tests of more general value. For instance, it is fairly obvious on looking at the photographs of Wilson Dam,[6] that, with all gates open, in even so large a river as the Tennessee, the velocity of approach is of some importance, whereas in a large reservoir behind a dam, say 300 ft. high, it is almost negligible. Therefore, the data obtained at a site such as the Wilson Dam do not afford information for the engineer building a high dam, although the individual gates (possibly only five or ten of them in this case), and the general shape of the weir are almost identical with the gates where the tests or observations have been made.

Although it is realized that, in some cases, the problem is difficult, yet it is thought that an attempt should be made to reduce and refer all coefficients to a "standard" or still-water condition.

The author, by the use of his "vertical velocity" and "vertical velocity area" curves, and his method of attack in integrating the area under the ellipse, has shown considerable ingenuity, and has presented the data very clearly and simply. He also deserves commendation for making the data which he has collected available for others.

---

[6] "The Design and Construction of Dams," by Edward Wegmann, M. Am. Soc. C. E., Eighth Edition, p. 630.

J. H. JONES,[7] ESQ. (by letter).—Whether or not one agrees with the conclusions drawn it is always an instructive pleasure to read a carefully executed investigation, and the equally careful analysis, such as Mr. Puls has made.

In carrying out his velocity measurements on such deep streams he has placed all hydraulic engineers in his debt, and British engineers who have few opportunities of dealing with large volumes of water find such researches especially valuable.

This discussion is not intended to present criticism, but is an alternative means of attack which may be of interest. The method has been crudely suggested in some notes by the writer, entitled, "The Standing Wave."[8]

Briefly, it consists of a denial of the usual theory, that, in the discharge through a large orifice, the velocity of any filament at a distance, z, below the free surface (see Fig. 4) is proportional to $\sqrt{z}$. Throughout the full depth at the *vena contracta* the velocity is sensibly uniform and proportional to $\sqrt{h_2 - cD}$, in which,

h_2 = depth from free surface to bottom of orifice.
c = coefficient of contraction.
D = depth of opening of gates (orifice).

The *vena contracta* is considered, since only there is the velocity of all filaments purely axial. Hence, the discharge from an orifice of length, L, depth, D, and with a head, H, is:

$$Q = c\,D\,L\,\sqrt{2g\,(H - c\,D)} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

TABLE 4.—COMPARISON OF DISCHARGE COMPUTED FROM TWO EQUATIONS.

D, in feet.	Q, IN CUBIC FEET PER SECOND.	
	By Equation (3).	By Equation (6).
1	726	785
2	1 474	1 540
3	2 202	2 260
4	2 952	2 960
5	3 692	3 630
6	4 404	4 255
7	5 076	4 860
8	5 692	5 450
9	6 308	5 980
10	6 978	6 460
11	7 657	6 980
12	8 378	7 350
13	9 130	7 760

The value of c is difficult to establish, but, in a sluice-way with a horizontal bed, it does not differ greatly from two-thirds. However, in the present case, because of the distortion caused by the blunt piers, 0.62 is the value used. Solving Equation (6) for various openings, D, under a constant head, H = 18 ft., on a 38-ft. gate-opening, the results are as listed in Table 4.

---

[7] Chartered Civ. Engr., Kilmarnock, Ayrshire, Scotland.

[8] *Engineering*, August 10, 17, and 24, 1928; see, also, "Notes on Sluice-Gate Discharge," *Engineering*, September 29, 1929.

It will be noted that these only differ appreciably from the values obtained by Mr. Puls in Table 1, for the highest values of D; but here the reduction in c, due to the piers and grooves, is least, the *vena contracta* is less definite, and the values of Q are thus greater than those calculated.

The writer has had the opportunity of seeing some German experiments which appear to confirm the fact that the value of c in Equation (6) tends to increase slightly as the gate-opening increases. This would have the effect of increasing the value of Q at the larger openings, bringing it into still closer agreement with the readings of the gauge.

It would appear that the sign and nature of the deviation of the writer's calculated discharges from those of Mr. Puls' formula, agree closely with the deviations shown by the readings on the Florence gauge.

Those who have facilities for sluice-gate measurements should try out this method as a check, so that engineers may possibly, as the information accumulates, be able to form some method of correlating values of c with the detail design of sluice-ways.

LOUIS G. PULS,[9] ASSOC. M. AM. SOC. C. E. (by letter).—The discussions of this paper manifest the keen interest of two engineers on the subject of spillway discharge. Their responses and inquiries are, no doubt, in the minds of many others. The writer does not deny the possibility of errors in the paper. There surely must be some, solely because it treats of quantities representing water discharge. Who can say truthfully how much water is flowing? Any treatise on the subject that departs from the old-fashioned "tin-can method" is surely destined to draw criticism. The paper was presented in a pioneer effort to supply a missing chapter in hydraulic science. Further investigation is needed, and it is to be hoped that others will venture into the field.

In regard to Mr. Pearce's discussion of the discharge with adjacent gates open, the writer wishes to emphasize that no effort was made in the paper to express a general law for the increment of discharge due to the influence of motion of the neighboring water mass. Referring to Fig. 2 it will be noted that two velocity measurements were made with adjacent gates closed. It should be stated that the increase is due to the reduction of end contractions or any other influences. In passing, it may be well to advance the theory that the elimination of friction between the water trains approaching the crest has a greater influence than the change of contraction between the two piers. No attempt was made in the paper, however, to invade the opinions or laws advocated by older authorities. The writer measured the increment of velocity at the depth stated and it is perfectly permissible to calculate the increment of head that would cause this velocity. Furthermore, it is logical to apply it as effective over the entire discharge area because, if any increment of head will increase the velocity at a point 2 ft. above the crest, it may also be expected to increase the velocity at higher elevations truly according to the fundamental law of falling bodies.

Mr. Jones has advanced an analysis that is commendable in its tendency to express a simple and usable law.

---

[9] Care, Hugh L. Cooper & Co., Inc., Kitchkas, Ukraine, Union of Socialistic Soviet Republics.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1771

COMPLETION OF MOFFAT TUNNEL OF COLORADO

By Clifford Allen Betts,[2] M. Am. Soc. C. E.

With Discussion by Messrs. J. Vipond Davies, R. H. Keays, A. A. Eremin, and Clifford Allen Betts

Synopsis

The 6.2-mile Moffat Tunnel through the Rocky Mountain Continental Divide, 50 miles west of Denver, Colo., was officially opened to railroad traffic on February 26, 1928.

The story of its conception many years ago and of the struggles since 1902 leading to its ultimate accomplishment has attracted widespread attention and has given the undertaking an unusually dramatic background. Difficulties encountered during construction due to caving ground and inflows of water led to the development of new tunneling methods which, combined with the unique Improvement District method of financing, added to the interest. Furthermore, the fact that this combination railroad and water tunnel embodied, at the time of its construction, the longest railroad tunnel in the Americas, in addition to a conduit of equal length, permitted the use of new and modern machinery and led to numerous refinements in standard equipment.

The history of the early stages of the work up to August, 1925, together with a discussion of the contract, has been published by the Society.[1] Inasmuch as that paper made no reference to the survey methods used in carrying alignment and grades over the range that rises to 12 000 ft. above sea level and 2 800 ft. above the tunnel, nor to the camps, these are described herein in some detail, together with other phases of the work which have developed subsequent to 1925.

---

[1] For a general description of the history, development, and progress of this undertaking to the summer of 1925, reference should be made to the paper by R. H. Keays, M. Am. Soc. C. E., entitled "Construction Methods on the Moffat Tunnel," *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 63. Published in April, 1930, *Proceedings*.

[2] Engr., U. S. Bureau of Reclamation, Owyhee Dam, Nyssa, Ore.

Soft ground encountered for a distance of 2½ miles in from the West Portal was the outstanding feature of the construction. The methods and machinery used to combat this obstacle, including the design of heavy timbering, steel reinforcement, concrete lining, and the use of the "Lewis Traveling Cantilever Girder" are among the tunneling developments described. Similarly, the notes on plant machinery and on costs of vital operations epitomize extensive studies.

Ventilation tests made at the Moffat Tunnel offered a unique opportunity for the correlation of data on air-friction factors for tunnels of mining size on which exhaustive information is available and on factors for standard railroad tunnels on which fewer tests have been made.

The construction of the Moffat Tunnel involved among other quantities (in round figures): 750 000 cu. yd. of excavation; 16 000 000 ft., board measure, of timber; 2 500 000 lb. of dynamite; 700 000 lb. of drill steel; 700 miles of drill holes; 28 000 000 kw-hr. of electrical energy; and 25 000 000 man-hours.

The final cost of work as shown by outstanding bond issues is $15 470 000, in addition to which approximately $2 000 000 revenue from camps, commissaries, interest on bank deposits, and other sources went into the work and into interest on the bonds during construction.

The ultimate object of the project is to make possible a transcontinental rail route through Denver; to insure adequate transportation facilities for Northwestern Colorado; and to provide an additional source of water supply for Denver and contiguous territory.

HISTORICAL RÉSUMÉ

Early Surveys.—When the final "holing through" blast, touched off by President Coolidge on February 18, 1927, broke down the 7-ft. granite barrier between the east and west headings of the Moffat Water Tunnel, it was said, as the Colorado and Utah officials joined hands, that this was the highest and lowest holing through of a tunnel in history; the highest because it was more than 9 200 ft. above sea level, and the lowest because the headings were 2 800 ft. below the crest of the Continental Divide.

For half a century this Divide, extending from the northern to the southern boundaries of Colorado, and beyond, had stood in the way of adequate transportation from the East to the West. A tunnel through the Rockies to provide a practical route had been first conceived in the early Sixties by Governor Gilpin when the Union Pacific Railroad Company began laying the first transcontinental track. It had been taken for granted that the route would pass through Denver, the largest center of the West. The late Grenville M. Dodge, Hon. M. Am. Soc. C. E., Chief Engineer of the Union Pacific Railroad, had surveyed all the passes from Laramie Canyon in Wyoming to the Arkansas River in Southern Colorado; but the unbroken barrier of the Divide rising to

12 000 ft. above sea level offered no suitable pass. On November 7, 1866, General Dodge's party barely escaped death in a blizzard near the site of the present East Portal of the Moffat Tunnel by turning their pack animals loose and going down the valley to the nearest camp on foot. This unfortunate experience did not show this site in a favorable light at the time, but eventually the process of elimination proved it to be the narrowest part of the Divide at an elevation suitable for railroading.

Surveys were run over this Rollins Pass again in 1867 for the Jefferson and Boulder County Railroad and Wagon Road and in 1869 by the Kansas Pacific Railroad Company. In 1880, a 2-mile tunnel site was surveyed and an open cut begun near Yankee Doodle Lake. This and some grading constitute the remaining evidences of the ambitious program of the Denver, Utah, and Pacific Railroad Company which, however, abandoned the work. Part of the grade was later acquired by the Chicago, Burlington, and Quincy Railroad Company, and in 1884-85 that Company ran surveys over Rollins Pass. A 3-mile and a 5-mile tunnel were projected; the east end of the latter being above and to the north of East Portal and the west end on Jim Creek west of James Peak.

Moffat Road Surveys.—Following this the Moffat Road ran ten lines in 1902, locating a 2.6-mile, a 4.1-mile, and a 5.8-mile tunnel. Plans went so far that bids were obtained on the 4.1-mile tunnel but, believing that the time required for construction would be prohibitive and that only the long tunnel would eliminate effectively the snow hazards, the road was built over Rollins Pass at Elevation 11 660, reaching Corona, Colo., on October 2, 1904. It was hoped that, pending construction of a tunnel, the scenic value of the line over the Rockies would compensate for the snow-fighting costs; but the advent of the automobile interfered, and the bitter fight against Nature forced the road into receivership by the time the Moffat Tunnel was begun in 1923.

David H. Moffat's historic fight against ruthless Nature and adverse railroad interests is an epic of vision, determination, and public-spirited generosity. His road reached Steamboat Springs on December 13, 1908, having cost $14 000 000 and later was extended to Craig, Colo. (255 miles from Denver).

Construction

Moffat Tunnel Surveys.—The Moffat Tunnel Act authorizing the construction of the Moffat Tunnel provided no funds that could be used for preliminary investigations, such as diamond drilling and surveys, prior to the sale of bonds. Sufficient private funds were made available for preliminary surveys, however, and thus the old Denver and Salt Lake Railroad (Moffat Road) survey lines were re-established and monumented. Upon geological advice that it would be advantageous to shift the West Portal location about 400 ft. south and the East Portal 75 ft. north, new portal sites were located by offsets from the old line. Timber cuttings for the new line were then started while the tunnel was being extended into the mountain on the offset lines.

The surveys for the Moffat Tunnel crossed the Continental Divide at a storm-swept point 12 000 ft. above sea level. There the crest of the Rockies flattens out, forming a plateau ½ mile wide which effectively prevented sighting toward each portal from a point on top. Consequently, it was necessary to establish three points on the Divide, so located that from the easternmost point the East Portal back-sight target could be seen (Fig. 1), while from the western point the West Portal terminal target was visible. These five points, forming the center line of the Water Tunnel, had to be placed in alignment at night because there was considerable refraction during daylight hours between the targets at elevations of 9 000 and 12 000 ft. This was attributed largely to the differing densities of the air and to temperature variations. Frequently, it exceeded 3 ft. on the 6-mile sight between the east terminal target and the Divide.

Lights placed behind paper or ground glass gave excellent results as sights especially after midnight when atmospheric conditions were most favorable. To permit accurate setting of points at night, a spirit-level head was mounted on a standard transit tripod with a painted metal target set vertically at one end so that a light could be placed behind and the target moved until it was on line. A hole in line with the pointed aperture of the target carried a plumb-bob string for setting points (using wind breaks). The final underground closure of 0.11 ft. after carrying the line 10 miles over the range and 6 miles underground substantiates the accuracy of this night work.

The line was carried underground through the Water Tunnel (which served as a pioneer bore), by setting up the transit on the terminal target, sighting on the skyline target on top of the Divide, depressing the telescope, and setting a point at the portal. From there the line could be carried in on special brass vernier bars, ¼ in. by 1 in. by 10 in., attached to plugs in the roof of the tunnel. These bars were accurately graduated in hundredths of a foot so that a brass vernier rider carrying a plumb-bob line could slide along the upper edge and permit readings to 0.0001 ft. These bars at about 500-ft. intervals not only made it easy to run independent lines and subsequently to compare the readings, but also facilitated adjustments where the ground and timbers were in motion. Caps moved several inches at points between 2 and 3 miles in from West Portal.

To determine the distance between portals a triangulation system was used, with a base line 3 756.587 ft. long, near West Portal, and eight stations on the mountains. Two U. S. Geological Survey, 300-ft. invar tapes having a coefficient of expansion of only 0.00000042 were used in measuring the base line, with corrections for sag and temperature. Supplementing the triangulation, slope measurements were made over the mountain with a 500-ft. chain, thus locating the profile. The horizontal distance through the tunnel was 1.75 ft. shorter than that computed.

Precise levels were run over the center line between the portals. These checked previous grades run independently over the Divide *via* the survey clearing and the Denver and Salt Lake Railroad tracks (31 miles). The grades checked within 0.345 ft. on holing through and were readily adjusted in the remaining enlargement (5 523 ft.).

Cross-sections of the Railroad Tunnel were taken with a "sunflower", mounted on a tripod, which consisted of a vertical aluminum disk graduated in degrees, beginning with zero at the top. Pivoted at the center was a rotating arm upon which a measuring rod could be placed to show simultaneously the distance and angle to a point on the tunnel wall. From the known positions of the "sunflower", sections could be plotted rapidly.

Work Begun by Commission.—Ground was broken for the Water Tunnel on August 25, 1923, following the decision of the United States Supreme Court validating the Moffat Tunnel Improvement District Law of May 12, 1922, whereby the Improvement District, of 9 000 sq. miles, including Denver, was created. Through its elected officers, the Moffat Tunnel Commission was empowered to assess real estate in the District, issue bonds, construct a railroad and water tunnel, and administer them when finished. This Colorado law provided for the making of contracts, not exceeding 99 years in duration, for the use of the tunnel up to the limits of its capacity for rail and water transportation as well as for power, telegraph, and telephone lines, at the same time prohibiting any monopoly or the sale of the property.

Pending the letting of a contract and in order to have the pioneer tunnel underground before the deep snows indigenous to the 9 000-ft. elevation arrived in October, the Moffat Tunnel Commission rushed the construction of camps at both East and West Portals. When a contract was awarded to Hitchcock and Tinkler, Incorporated, on September 20, 1923, the camps were under roof, the West Portal Water Tunnel had been driven in 32 ft., and the open cuts were well advanced.

West Portal Camp.—Both East Portal and West Portal camps were constructed in 1923 while machinery was being delivered and the excavation of the tunnel started. The camp sites were so different in appearance that it was frequently said that the portals hardly seemed to be in the same mountain. This difference persisted as the headings extended underground. West Portal, the first camp completed, was surrounded by heavily timbered slopes beneath which lay the soft rock of an extensive fault zone, while East Portal had more barren and precipitous surroundings (Fig. 1), showing the solid gray granite which extended to the center of the Divide.

While the camp site at West Portal was being cleared of heavy pine, and the buildings were being constructed, the men were quartered at Wood's saw-mill near-by. The water supply for this site was taken from Buck Creek and piped by gravity to camp and plant buildings, provision being made for fire hydrants in the camp streets. Large septic tanks were used for the sewerage system. The ten 30-man bunk-houses, 20 by 50 ft., with rooms accommodating two men each and allowing 250 cu. ft. of air per man, were built 75 ft. apart to reduce the fire hazard.

These houses were wired for electric lights, heated by pipeless hot-air furnaces, and built with paper between the inside and outside sheetings. A building was provided for the Moffat Tunnel Commission Engineering Staff and one for the Contractor's organization. Later, these were augmented by eight 4-room cottages for engineers and foremen with families and by twenty-two 2-room cabins for miners with families, preference being given by seniority. The cottages rented for $25 per month and the cabins for $15, which practically amortized the cost of construction. Many families lived in privately owned houses near the portal camps.

A mess hall seating 250 and serving as many as 40 000 meals per month, at a cost of about 42 cents each, was constructed between the bunk-houses and the

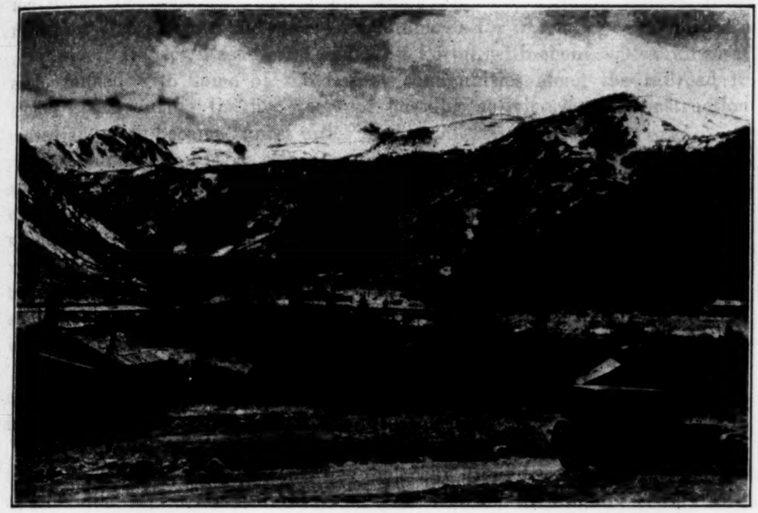


FIG. 1.—EAST PORTAL, MOFFAT TUNNEL, LOOKING WEST FROM BOULDER CREEK. CAMP BUILDINGS IN CENTER AT FOOT OF MOUNTAIN. PARALLEL MARKS OF SURVEY LINE APPEAR STRAIGHT AHEAD FROM WOODED SLOPE.

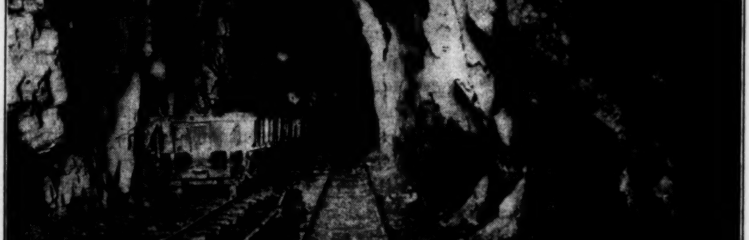


FIG. 2.—TYPICAL RAILROAD TUNNEL THROUGH GRANITE, SHOWING EQUIPMENT. LATER, 36-INCH GAUGE TRACKS WERE REPLACED BY BALLASTED STANDARD SINGLE-TRACK LINE.

[illegible]

[illegible]

portal, as was also a recreation hall containing a commissary, post office, reading room, pool hall, and auditorium which could be used for dances, prize fights, moving pictures, and other entertainments for which the camp gained a reputation. In fact, East Portal and West Portal, with about 500 workmen at each, ranked with some of the largest communities along the railroad in money spent during the five years of bustling activity before construction ceased and they became "ghost cities".

The laundry, equipped with rows of stationary tubs, shower baths, toilets, and plenty of hot water, was the central wash house, there being no tubs in camp. As a preventive of pneumonia, which at that altitude is pernicious, a change house was built at each portal with lockers and showers so that workmen coming off shift could enter the building and change to dry clothes before going out into the cold. Wet clothes were attached to wire rope hoists and raised out of reach overhead to dry, the free end of each rope being secured inside the owner's locker.

Grade schools accommodating 60 pupils were provided for the camp children at each portal. The Moffat Tunnel Commission furnished the buildings, and the County School Districts, the teachers.

East Portal Camp.—The camp at East Portal (Fig. 1) was planned similar to the West Portal camp, except that the rocky ground led to the installation of individual septic tanks for the houses instead of ditching for sewers to a central disposal plant.

Plant Buildings.—The plant buildings at each portal were grouped closely around the portals and connected to the Water Tunnel by snowsheds so that the severe winter weather, that frequently extends into June at this locality, would not handicap operations.

A steel sharpening shop opened directly off the tunnel. Beyond this was the machine shop with lathe, drill press, grinders, welding outfits, and all tools required for large or small repair work.

The power house received 2 300-volt current transformed from the 44 000-volt, 3-phase, 60-cycle transmission line from the Boulder Canyon Plant of the Public Service Company of Colorado. Power consumption was planned to keep the maximum demand as low as possible, usually about 1 800 h.p., with the result that the cost averaged 1 cent per kw-hr. for a total use of about 28 000 000 kw-hr.

Comparatively few power interruptions occurred on the power line to East Portal, but the 7-mile line between portals, over the Continental Divide, required considerable maintenance during winter months. Tubes of ice formed on the wires by snowdrifts and sleet storms and then, swayed by the high gales, broke off insulators or poles. This usually occurred at the highest part of the line where iron wire replaced copper for strength. On one occasion a Mexican laborer tripped across a section of the power line that was buried in snowdrifts and was burned beyond recognition.

Adjacent to the power house was the electricians' shop and the battery recharging station for the storage batteries used on the 4-ton locomotives. The warehouse, primer house, and carpenter shop were grouped near-by.

At West Portal a timber-framing shed was equipped not only with the usual cut-off and band-saws but also with a timber dapping machine consisting of a dado-head saw that could be drawn across the timbers, cutting to desired depths and widths.

Among the items included in the plant list used in advertising the machinery for sale at the conclusion of the job were the following, with their costs, f. o. b. job:

Rolling stock	$213 171.00
Air shovels and mucking machines	62 573.50
Drills	53 541.08
Concrete mixers and pneumatic placing machine	25 653.83
Lewis girders	26 782.00
Motor generator sets	20 848.96
Pumps	8 612.00
Blowers	14 279.00
Equipment in shops and power houses	103 000.00
Miscellaneous electrical equipment	12 000.00
Hoists and jacks	11 795.67
Rail	30 180.00
Wire	54 322.00
Pipe	58 134.89

The "Inventory of Plant Machinery, Camp Buildings, and Equipment" showed a summary of:

Total plant machinery	$739 499.01
Total camp buildings (East Portal)	100 096.48
Total camp buildings (West Portal)	124 926.18
Total camp equipment (East Portal)	13 200.00
Total camp equipment (West Portal)	15 800.00
	$993 521.67

Geological Conditions Encountered.—From the beginning the ground at West Portal presented difficulties. The East Portal headings, on the other hand, passed through comparatively uniform grey metamorphic granite (Fig. 2), with some quartz monzonite and injection gneiss seams of consistently similar dip and strike. In all, there was less than 1 500 ft. of seamy ground requiring support east of the apex and at no place were the pressures great. This was the character of formation on which the plans were predicated.

It is interesting to note that this end of the tunnel was completed six months ahead of schedule and practically within the original estimate. The semi-annual financial statement of the Moffat Tunnel Commission of January, 1925, before costs had been greatly increased by the unforeseen difficulties, showed the cost of excavation in the headings to be $10.41 as compared with $17.00, the contract price; and the cost of enlargement excavation, $4.80 as compared with $4.70 bid. (See Appendices I and II.)

The fact that the east headings overcame the half-mile lead that the West Portal had acquired by starting first and eventually made a mile greater progress indicates the difference in rock at the two ends. Furthermore, nearly twice as much dynamite per yard (11 lb.) was required for the East Portal rock as for that at West Portal.

Geologists had predicted less than 2 000 ft. of questionable rock that might require support; no funds had been allowed for preliminary diamond drilling. As a result the 2½ miles of fault zone with seams and dikes not only increased

the cost of the project, but required more time, as is graphically shown on the progress chart, (Fig. 3). Subsequent geological studies made from the bore within the mountain indicated that the hidden fault zone, which caused the shattered rock at West Portal, extended for such a long distance along the Divide that to have avoided it by shifting the tunnel location would have been impractical.

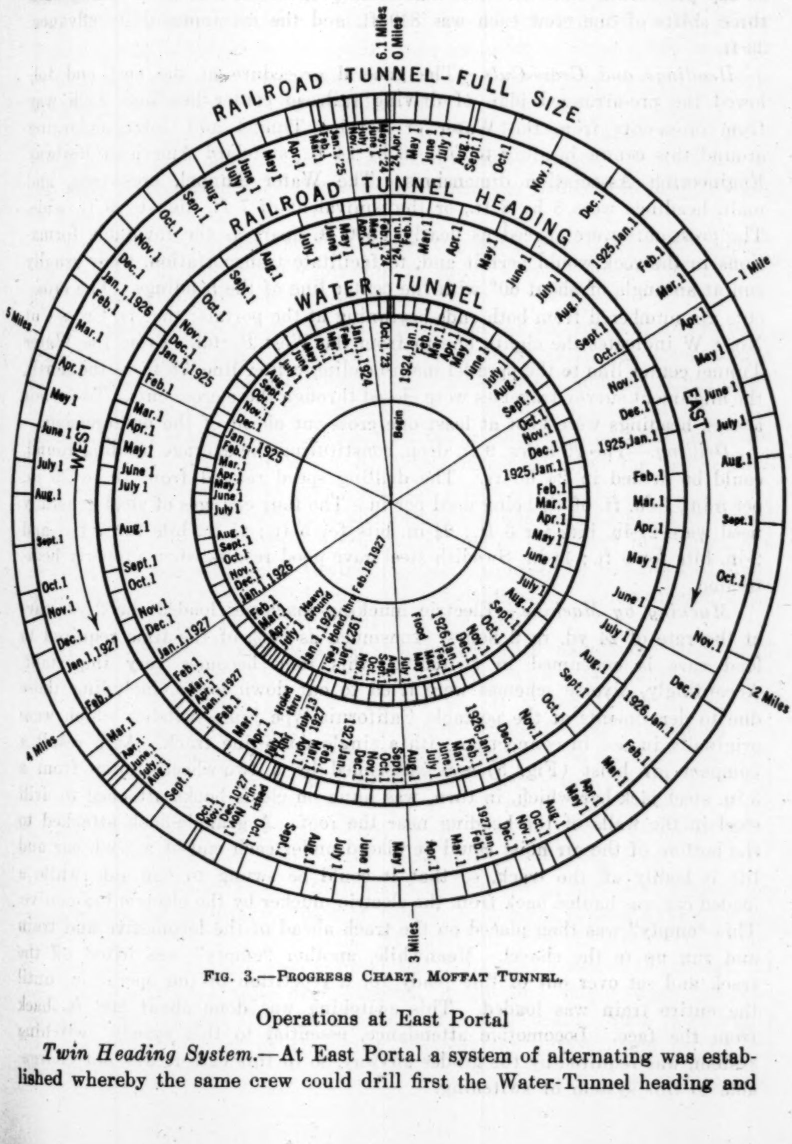


FIG. 3.—PROGRESS CHART, MOFFAT TUNNEL.

Operations at East Portal

Twin Heading System.—At East Portal a system of alternating was established whereby the same crew could drill first the Water-Tunnel heading and then, while that was being mucked out after shooting, could move over to the Railroad-Tunnel heading and proceed to drill there without interfering with the mucking crew. In this way three shifts of about seventeen men each could usually drill, shoot, and muck three 8-ft. rounds in 24 hours, or a full round each shift. Under this system the maximum advance made during a 30-day period was 1 583 ft. The maximum progress in a single heading with three shifts of one crew each was 849 ft. and the maximum daily advance, 33 ft.

Headings and Cross-Cuts.—The general procedure at the east end followed the pre-arranged plan of driving railroad center headings each way from cross-cuts from the Water or Service Tunnel and, later, enlarging around this center heading to full 16 by 24 ft., standard American Railway Engineering Association dimensions. The Water Tunnel, cross-cuts, and main headings were 8 by 8 ft., or the equivalent of 7 ft. high by 9 ft. wide. The cross-cuts were spaced as nearly 1 500 ft. apart as the favorable formations in the rock would permit and, to facilitate transportation, were usually run at an angle of about 60° with the center line of the headings. The cross-cuts are numbered from both ends beginning at the portals; that is, Cross-Cut No. 8-W indicates the eighth one in from the West Portal. From the Water Tunnel center line to the parallel main-heading center line, 75 ft. to the north, the alignment survey traverses were closed through these cross-cuts. To permit access, headings were kept at least one cross-cut ahead of the enlargement.

Drilling.—The 26 holes, 9 ft. deep, constituting the average heading round, could be drilled in 2½ hours. The drilling speed ranged from 10 to 25 in. per min., 1 cu. ft. of air being used per in. The four changes of steel generally used were 2⅜-in. bits for 3 ft.; 2¼-in. bits for 5 ft.; 2⅛-in. bits for 7 ft.; and 2-in. bits for 9 ft.; 1¼-in. Swedish steel gave good results when properly heat-treated.

Mucking by Machine.—Electric mucking machines loaded the 2-yd. cars at the rate of 24 yd. in 2 hours. Inasmuch as 60% of the time required to load cars is consumed in switching, this item becomes very important. Accordingly, diverse schemes were tried to cut down losses, including those due to derailments at the portable California-type jump-switches which were originally in use in connection with a single main-line track. As a result a compact air hoist (Fig. 5) was suspended by a two-wheel trolley from a 3-in. steel jack bar which, in turn, was hung on clevis hooks attached to drill steel in the walls of the heading near the roof. A grapple-hook attached to the bottom of the air hoist could be placed under each end of a 2-yd. car and lift it bodily off the track, so that it could be swung to one side while a loaded car was hauled back from the electric mucker by the electric locomotive. This "empty" was then placed on the track ahead of the locomotive and train and run up to the shovel. Meanwhile, another "empty" was lifted off the track and set over out of line ready for a repetition of the operation, until the entire train was loaded. This switching was done about 200 ft. back from the face. Locomotive attendance, essential to this speedy switching system, was required by the grades anyway, so in this case it was not chargeable to this system of switching.

Hauling.—The high-sided 2-yd. cars used on the 24-in. track (40-lb. rail) in the headings were suitable for machine loading and not adapted to hand mucking. In order to relieve these cars of the long trip out on the dump and to release them for the many heading operations which sometimes numbered twenty or more and made heavy demands upon transportation, transfer docks were provided where muck from the narrow-gauge cars could be dumped into the 4-yd., 36-in. gauge cars on the railroad tunnel level, 8 ft. below the Water Tunnel grade. Sidings for "empties" were installed at every other cross-cut so that dispatching of transportation might be more flexible. Hauling costs increased perceptibly with the distance in.

Underground Accommodations.—A "Harvey House" (lunch room) in a niche in the rock more than a mile underground permitted the workmen to rest after being served hot coffee with the lunch they had selected and brought in with them.

"Drill doctor" benches and electricians' cupboards were distributed through the headings at convenient cross-cuts in order to save delays. Mine telephones were also installed at cross-cuts.

Enlargement in Hard Rock.—At East Portal the enlargement of the Railroad Tunnel center heading from 8 by 8 ft. to 16 by 24 ft. was accomplished for the most part in one operation by the ring-shooting method. Rings of 27 holes each were drilled 4 ft. apart, the direction and spacing of the holes normal to the center line of the tunnel being controlled by steel templates attached to the drill housing or by offsets measured from the center line. Drilling was carried far in advance of the bench.

In shooting, the lower half of the excavation was kept about 16 ft. ahead of the upper ring. As a rule, a round consisted of two lower half rings, one without, and one with, delay electric exploders, followed by two top half rings with "delays". "Overbreak" in the railroad enlargement approximated 20%, while, in the Water Tunnel, it averaged 50 per cent.

The air shovel having a 1⅓-cu. yd. bucket with manganese steel teeth consistently averaged 24 ft. per day (two shifts), with a maximum of 53 ft., and maximum monthly progress of 1 002 ft. It moved 11 000 cu. yd. per month and remained underground 18 months without being overhauled.

In 1925, a crushing and screening plant was installed on the East Approach dump and 2½-in. ballast as well as sand and aggregates for concrete and "gunite" was produced from the tough granite in the eastern part of the tunnel. Altogether, more than 66 000 cu. yd. were treated at about 50 cents per yd., including cost of the plant, operation, and maintenance.

Gunite.—Through ground that would decrepitate, or air-slack, upon exposure, a coat of "gunite" (1 part of cement to 3 parts of sand), 1½ to 3 in. thick, served as an ideal protection. At about one-tenth the cost of timbering, it accomplished the same results providing there was no weight to support. It was particularly valuable over diagonal seams that might wash out and release blocks of rock after the air had softened the seam. Such spots would require a long stretch of timbering, whereas the gunite could be applied locally.

Beginning in 1925 gunite was used extensively covering thousands of square feet of rock surface at a cost for a 2-in. coat of 7 cents for labor plus 8 cents for material, or $8 to $14 per lin. ft. of tunnel as actually placed. About 20 000 sq. ft. was a good month's work. Much of the rock east of the apex was thus protected, only about a mile being left exposed.

Water Inflows.—Water was the greatest handicap encountered at East Portal. On February 18, 1925, a water seam was cut in the Water Tunnel 8 050 ft. in from East Portal.[3] This connected directly with Crater Lake, 1 300 ft. above, and produced 1 800 gal. per min. until silt from the lake bottom clogged the seam. Again, in October, 1925, water seams were encountered, which produced several hundred gallons per minute and materially retarded progress. As new seams were cut, the back seams would dry up, so that the headings, which were then on the down grade beyond the apex (Station 2536+54), required pumps.

The main seam 1 800 ft. below Ranch Creek was tapped on February 28, 1926. The "graveyard shift" coming in to muck out the heading hung up their electric light cluster and returned to Cross-Cut No. 10-E, 300 ft. back, for tools. When they returned to the heading it was noticed that the lights had apparently been moved. By this time the water, which had entered from below the muck-pile and had been slowly moving it with respect to the lights, burst through. It carried the pile 20 ft. down the tunnel and brought in 900 yd. of fine materials which were strewn for 200 ft. down the track.

The mucking machine and tools were moved to Cross-Cut No. 10 before the water reached the waists of the men, who then retreated to the apex 1 250 ft. away and installed pumps. Meanwhile, the water ran through the cross-cut and down to fill the Water Tunnel heading. Within five hours both headings were filled and water started to flow eastward over the apex through the Water Tunnel, the Railroad Tunnel having been dammed with sand-bags to protect enlargement work there. When the water reached Cross-Cut No. 8, it was diverted into the Railroad Tunnel east of the bench operations.

For a time a weir built at Cross-Cut No. 8 registered 2 500 gal. per min. and the tracks were covered with water, interfering with the electric locomotives; but within a week, the flow having subsided to 1 500 gal. per min., the headings were unwatered and work was resumed. In unwatering the headings, pumps and pipe were mounted on flat cars (Fig. 4) and backed toward the water, pumping under heads as high as 10 ft. From March 8 to April 8, the flow was 1 500 gal. per min.; in May, the flow decreased to about 600 gal. per min., which had to be pumped until the headings were holed through in the Water Tunnel and the benches completed in the railroad bore.

Other flows, totaling 450 gal. per min., were intercepted between Cross-Cuts Nos. 13 and 14. These were in looser rock and had the effect of softening the adjacent ground, thereby interfering not only with the holing through of the headings, but also with the excavation of the last 100 ft. of bench, which happened to come near the same location.

---

[3] "Construction Methods on the Moffat Tunnel," by R. H. Keays, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 68.

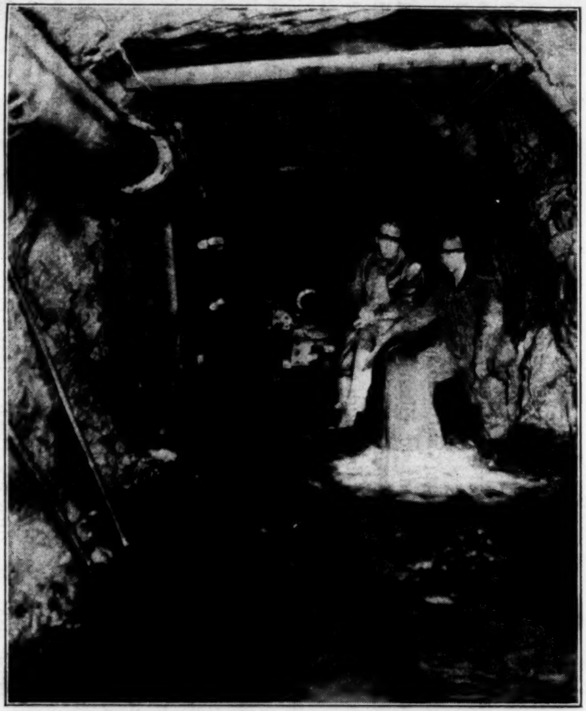


FIG. 4.—UNWATERING HEADING. PIPE LINE DISCHARGING AT APEX WHENCE WATER FLOWED BY GRAVITY TO PORTAL.

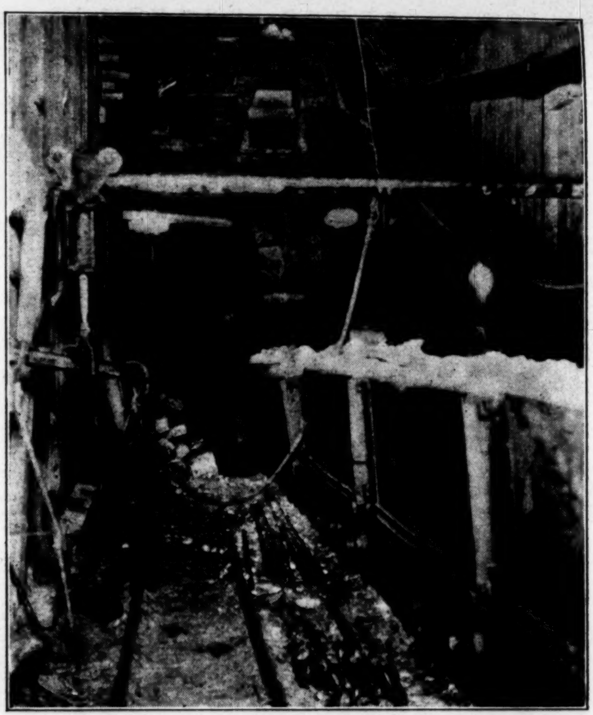


FIG. 5.—OPERATION OF LEWIS GIRDER DISCHARGING MUCK INTO CARS. AIR-LIFT CAR SWITCHER SHOWN IN FOREGROUND.

At the completion of the tunnel a fairly constant flow of about 600 gal. of water per min. was leaving *via* the West Portal drainage gutters and about 30 gal. per min. out of East Portal.

Operation at West Portal

General Plan.—At West Portal, the Water Tunnel functioned as a pioneer tunnel, advancing far ahead of the Railroad Tunnel enlargement so that heading, widening, and enlargement operations could be carried on simultaneously through the numerous cross-cuts. In this way as many as twenty attacks were made at different points at the same time, greatly expediting the work through the hazardous ground.

As at East Portal, all piping, wiring, and heading transportation were carried through the service tunnel, leaving the enlargement operations free of interference from these sources. The piping system included 7¾-in. casing for high-pressure air, 3½-in. water line, 12-in. ventilating line, and, in some places, pump discharge lines. The wiring provided for a 2 300-volt, 3-phase, 60-cycle, lead-covered cable supplying motor-generator sets at strategic points in the Water Tunnel; 250-volt direct-current wires for trolleys, mucking machines, and pumps; a 250-volt, 50-ampere "shooting" line; a 110-volt lighting line; and a telephone line.

Where pressures were encountered, top headings were followed by widenings for roof segments which, in turn, were followed by the excavation of the bench below the wall-plates either in one or two lifts. At some places west of the apex in rock that appeared solid, center headings were attempted and the Water Tunnel grades were accommodated to this 8-ft. change. However, such sections almost invariably required lining when opened up to the full size of the railroad bore. In changing from center to top headings, gradients as high as 5% were negotiated by the muck trains, but 2% was the maximum for machine mucking.

Bench Excavation.—The bench excavation presented the most difficult problems through the soft ground. Usually by the time the bench crew of fifteen men started work the large rock area had been exposed long enough to make it treacherous if it was not actually developing extreme pressure, tending to force in the side posts and allowing the roof to settle.

The cycle of operations on the bench ranged from 12 to 23 hours, as a result of which a "call shift" was instituted. The drilling of thirty to forty horizontal holes 20 ft. deep proved more satisfactory than drilling vertical down holes ahead in an attempt to save time.

Loss of Life.—During the early part of 1925 difficulties reached a crisis. At the same time that water was hindering progress at the East Portal, crushed, folded, and faulted rock was causing delays in the West Portal headings and widenings. Small cave-ins occurred in the headings and crushing wall-plates threatened to let down the roof timbers in the enlargement as unprecedented pressure developed.

One serious cave-in occurred in the Railroad Tunnel 175 ft. east of Cross-Cut No. 9 (2.3 miles from West Portal), on July 30, 1925, when six workmen of the day-shift timber crew were killed by 100 tons of rock that fell without warning from the north side of the roof as timbers were being placed. Characteristic heroism was displayed by the miners who volunteered to a man for the rescue work which was organized in case any of the men were buried alive. The coroner returned a verdict of "unavoidable accident".

In all, twenty-eight men made the supreme sacrifice in putting through the Moffat Tunnel—a remarkable record considering the hazardous nature of the formations encountered.

Lewis Traveling Cantilever Girder.—In order to cope with the emergency the first Lewis traveling cantilever girder, originated by George Lewis, General Manager, was installed at the West Portal bench near Cross-Cut No. 2 in March, 1925. This invention (Figs. 5 and 6) had numerous advantages, foremost among which was mobility. Designed to support the roof timbers and wall-plates (as a cantilever resting on, and extending back from, the heading) while the bench was being excavated and posts were being set, this machine successfully held the weight and increased the speed of progress. It is valuable in soft-ground tunnels of large size and can be used for either heading and bench or pioneer tunnel operations.

Four of these girders were used on lined sections of the Moffat Tunnel extending two miles or more, and their effectiveness has been tested through soft running ground of slicken-sided schist and gneiss as well as through blocky granite. On one occasion, Girder No. 1 caught a cave-in, protecting the workmen beneath. After tons of muck had been removed, the long girders, which had been bent downward several inches at the end, returned to their original alignment.

Girder No. 2 installed in October, 1926, was equipped with a conveyor belt, 30 in. wide, operating at 30 ft. per min. by means of a 50-h.p. electric motor. Being reversible, this conveyor could carry muck from the heading back through the center of the girder and discharge it through a 14-ft. chute into 4-yd. cars behind the bench (Fig. 5), or it could carry timbers and supplies from the bench ahead to the headings. The other three girders were without conveyors inasmuch as they were used for benches where widening had been handled through cross-cuts ahead. A description of Girder No. 2 (Fig. 6), therefore, will cover the others.

Two plate girders, 42 in. deep by 65 ft. long, spaced 6 ft. apart with rigid cross-braces of structural angles, which also served as the frame for the conveyor rolls, constituted the backbone of the machine. Cross-arms extending at right angles to the center line of the girder and supported beneath it by steel stirrups could be moved out under the wall-plates and jacked tightly up against them, using 15-ton screw-jacks built with the hanger and operating through a lever and rocker-arm. There were twelve cross-arms at the rear, or bench end, of the girder to support the roof and four at the forward end to keep it in place against the up-thrust when the load was applied at the rear end. Jacks against the roof were sometimes used for the same purpose.

In addition to the twelve cantilever arms beneath the wall-plate there were two that rested upon the top of the wall-plate, converting the cantilever into

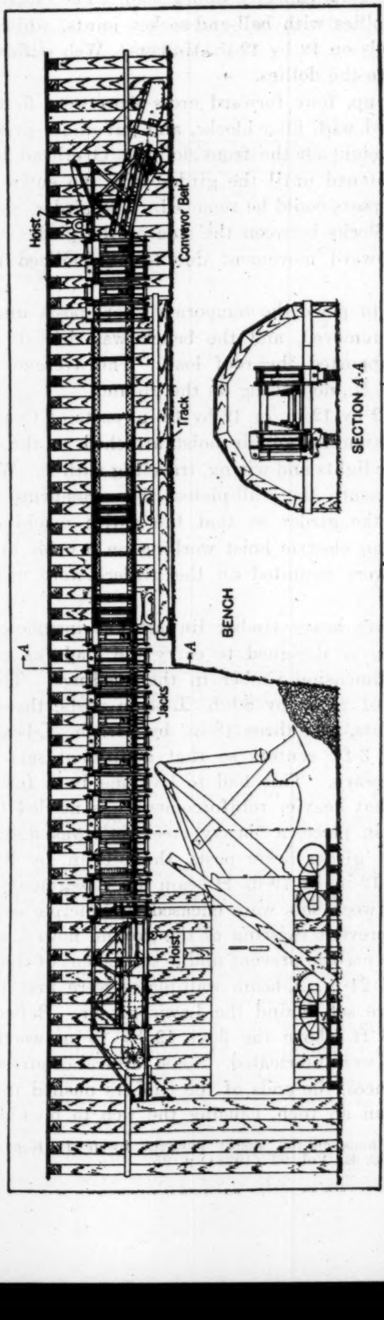

Hoist
Conveyor Belt
Track
SECTION A-A
A
A
BENCH
Jacks
Hoist

FIG. 6. Lewis Traveling Cantilever Girder: Elevation Showing Support on Bench, Jacking Under Free Arch to Permit Unrestricted Mucking, and Conveyor System

a beam and increasing its capacity under load. The entire machine rode on two pairs of steel dollies with ball-and-socket joints, which traveled along a track of steel channels on 12 by 12-in. timbers. Web stiffeners reinforced the ½-in. web-plates above the dollies.

In making a set-up, four forward cross-arms were first rolled out under the wall-plate, wedged with filler blocks, and put under pressure by the jacks, thus throwing the weight on the front dollies. Overhead jacks or rear cross-arms were next tightened until the girder took the entire roof load so that the temporary short posts could be removed. The girder, as designed, weighed about 200 000 lb. Blocks between the side of the girder and the wall-plates counteracted any inward movement due to unequalized roof and side-wall loads.

With the girder in place the temporary short posts under the wall-plates at the bench were removed, and the bench was drilled, shot, and mucked while the girder supported the roof load. The average progress per shift varied from 12 to 24 ft., depending on the ground.

Following this, 12 by 12-in. or 12 by 18-in. posts of Oregon fir were pulled up into plumb position by an air hoist attached to the end of the girder (Fig. 6) and, like the lights and wiring, traveling with it. When these had been wedged in place beneath the wall-plates, the cross-arms were released and pulled back under the girder so that the entire machine could be moved ahead by means of an electric hoist working on a cable attached to the timbering ahead. Blowers mounted on the girders were useful in ventilating the heading.

Steel Sets.—Where heavy timber lining[4] was insufficient, steel sets with arches and inverts were designed to carry the loads as roughly determined by the failure of dimension timber in the headings. The first, or lighter, steel sets consisted of 24-in. by 80-lb. I-beam posts, three 20-in. by 65.4-lb. I-beam roof segments, and three 18-in. by 54.7-lb. I-beam invert members (Fig. 7) placed on 3-ft. centers so that additional sets could be "jumped in" between if necessary. This had to be done in a few instances. When it became evident that heavier reinforcement was needed to hold the ground until concrete was in place, a "heavy steel set" was designed consisting of two 20-in. by 113-lb. girder beam posts, three 14-in. by 100-lb. H-beam arch members, and three 12-in. by 79-lb. H-beam invert segments. The 10-in. channel collar braces between sets were encased in concrete as soon as they were bolted, in order to prevent twisting as the ground flowed, and reinforced pre-cast foot-blocks were used to prevent undue settlement of the posts (Fig. 7).

Roof arches and 24-in. I-beam wall-plates were first placed, followed by the posts which were set behind the Lewis girder. Between the posts at a distance of about 8 ft. above the floor 12 by 18-in. wooden spreaders were wedged until inverts were fabricated. Some failures occurred in these spreaders and, in a few instances, the ends of the timbers pushed in the flanges of the steel posts more than an inch, causing the web to buckle and necessitating replacement of the post. In places, the floor squeezed up several inches a day, requiring repeated excavation before being finally filled with concrete.

---

[4] "Construction Methods on the Moffat Tunnel," by R. H. Keays, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 95.

Costs of timber lining and of steel lining are given in Appendices III and IV.

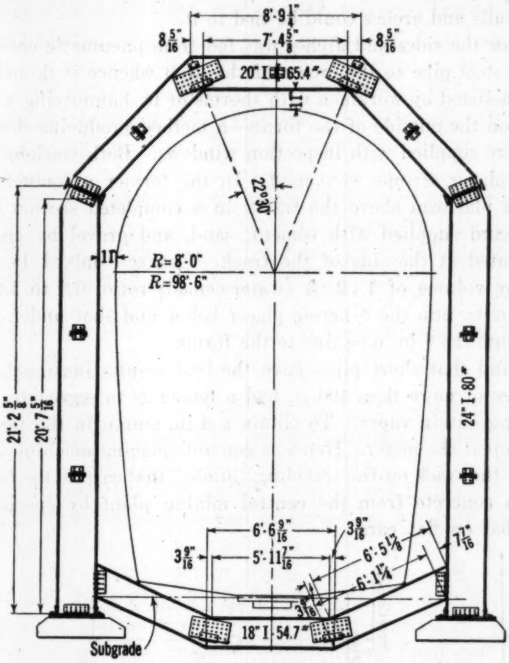


FIG. 7.—STEEL SET FOR TUNNEL LINING—LIGHT TYPE.

Concrete Lining.—To minimize this squeezing, which increased with exposure of the soft rock to air and moisture up to a point where arching created a pseudo-equilibrium, concrete was placed in the inverts as soon as possible after the steel. Reinforcement of 30-lb. rails and 1-in. bars was placed between the 5½-ton steel sets so that the entire lining was consolidated. Folding wooden forms in 4-ft. sections, lined with sheet zinc and suspended by block and tackle from the timber spreaders, were held down by bracing until the concrete set, with the result that none of the concrete was ruptured during the early setting stages. Cement which gave early compressive strength of more than 2 000 lb. per sq. in. in 48 hours and 4 500 lb. in 7 days, was used.

In one of its reports to the Moffat Tunnel Commission, the Board of Consulting Engineers called attention to the tendency of the weak rock, if held securely, to reach a constant distributed bearing, insuring permanent safety. Instrumental observations made on the lining to determine the movement, if any, had shown this to be the case.

Following the lining of the inverts, traveling steel forms, mounted on a timber "jumbo" carriage, were moved along the tracks on the finished invert.

When jacked into position the bottom of these forms was fastened to bolts previously embedded in the upper shoulder of the invert near the keyway joint. In carrying the invert up to this joint, 4 ft. above the center-drain flow line, reinforcing rods were left protruding upward so that the steel for the side-walls and arches could be tied to it.

Material for the sides and arches was fed from pneumatic concrete placers through 8-in. steel pipe to the crown of the arch whence it flowed behind the forms, being assisted by workmen with shovels or by hammering with one-man air hammers on the outside of the forms—a method producing dense concrete. The forms were supplied with inspection windows. Both stationary and moving concrete placer set-ups were used. In the former a concrete mixer was mounted on a platform above the tracks in a completed section of the Railroad Tunnel and supplied with cement, sand, and gravel by endless bucket conveyors located at the side of the track. Concrete mixed 1½ min. in the proportions by volume of 1 : 2 : 3 (water-cement ratio, 0.8 to 1.0) was then poured by gravity into the concrete placer below and shot under 80 to 110-lb. pressure through the 8-in. pipe line to the forms.

It was found that short pipes gave the best results inasmuch as concrete shot a distance of more than 100 ft. had a tendency to segregate, some of the cement blowing out in vapor. To obtain a 4-in. slump in the forms required 6 to 7-in. slump at the mixer. Hence, a concrete placing machine was mounted to one side of the track on the traveling "jumbo" that carried the forms and was supplied with concrete from the central mixing plant by special side-dump bodies mounted on flat cars.

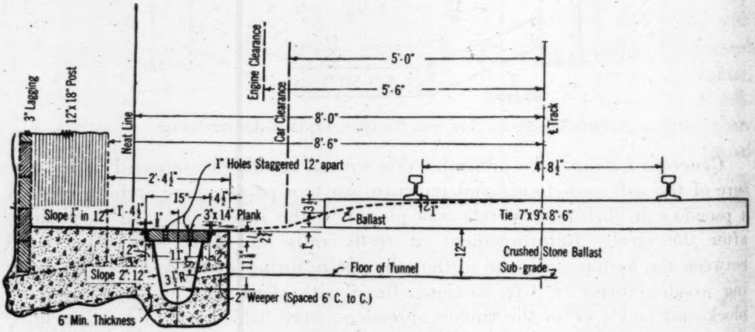

3" Lagging
12"x 18" Post
Neat Line
Engine Clearance
Car Clearance
5'-0"
5'-6"
8'-0"
8'-6"
℄ Track
1" Holes Staggered 12" apart
2'-4½"
15"
4½"
4'-8½"
Slope ¼" in 12"
1'-4½"
1"
3"x 14" Plank
3"
Ballast
Tie 7"x 9"x 8'-6"
2"
1½"
2½"
2"
11½"
12"
Crushed Stone Ballast
Slope 2": 12"
3½"R
Floor of Tunnel
Sub-grade
2" Weeper (Spaced 6' C. to C.)
6" Min. Thickness

FIG. 8.—DRAINAGE GUTTERS FOR RAILROAD TUNNEL, CROSS-SECTION SHOWING RELATION TO WALL AND TRACK.

In all, 916 lin. ft. of steel lining was buried in concrete and a total of 12 000 yd. of concrete lining was used, including that around the wooden sets. The thickness of the concrete ranged from 1 to 5 ft. Throughout the timbered sections allowance was made for future lining of concrete by setting the posts back. The far-sighted policy of concreting the sections showing greatest need each year, if followed out consistently, will result eventually in a complete permanent lining—a desirable and, in the long run, an economical condition. The cost of concrete lining will be found in Appendix IV.

Drains.—Concrete gutters through the timbered sections (Fig. 8) served the dual purpose of protecting the timbers from the deteriorating and undermining effects of water and of anchoring the posts against "kicking in". A capacity of about 3 000 gal. per min. was provided in the gutters, work on which was begun in April, 1926, and carried on by separate crews following the lining operations.

Center drains through the concrete-lined sections were connected with the concrete side gutters by transitions. Tile drains, made to order, were laid with open joints along the south side of the tunnel through the solid rock sections east of the apex. These were entirely covered with ballast.

Spark Protection.—A canopy 10 ft. in width, of 1-in. by 4-in. California redwood, tongued and grooved, was nailed to the roof segments through the timbered sections to prevent live sparks from lodging in the cracks and starting fires. This type of fire protection has been used on timbered tunnels along the Moffat Railroad.

Refuge Bays.—Twenty-one niches, or refuge bays, were left in the south wall of the railroad tunnel at cross-cuts for the convenience of maintenance crews.

Water Tunnel

Steel Plate.—As the excavation in the Railroad Tunnel progressed the timbering in the Water Tunnel, 62 ft. away, showed movement and distress, making it imperative that this section be protected from cave-ins. Accordingly, 1 485 ft. of the water bore east of Cross-Cut No. 9-W was enlarged to the size that will ultimately be required for the transportation of the Fraser River water supply to the eastern slope. This was lined with corrugated pressed steel liner plates (Fig. 9), inside of which was a 4-in. coat of gunite, with allowance for additional concrete if needed between the gunite and the 10-ft. inside diameter. The liners were cold-pressed from steel plates by dies made to give twelve segments 36 in. long by 16 in. wide by ¼ in. thick and with a 2-in. bolting flange rounded at the corners. This feature, with the corrugations, gave the required strength to units that weighed 50 lb. and could be readily placed by one man. Added strength was obtained when required by splicing ¼-in. segmental stiffener bars between the flanges.

The fact that a full circle of these plates could be bolted and wedged in place to line and grade without disturbing enough of the heavy ground to create a hazard, proved a distinct advantage. Furthermore, excessive excavation was obviated as the steel virtually became reinforcing. Holes were provided at intervals of a few feet so that the packing behind the steel plates could be grouted under pressure. Having thus taken the weight of the ground on this circular steel lining a protection of gunite (1 part cement to 2 or 3 parts sand) was placed to a uniform depth of approximately 4 in.

Loads estimated at 129 000 lb. per lin. ft. were supported by this type of lining without deformation in the steel or cracking in the cement.

Future Use.—The Moffat Tunnel Law made it mandatory upon the Commission to provide for transportation of water from the Colorado River Basin on the western slope to the dry eastern slope, the purpose being to increase the available water supply for the City of Denver. Therefore, plans were made for a parallel bore as close as practical to the Railroad Tunnel that would serve as a service tunnel to speed up construction and later could be enlarged to carry 1 000 cu. ft. per. sec. of water—the capacity required to divert economically 100 000 acre-ft., the mean run-off from the 97 sq. miles of the Fraser River Diversion Project upon which the City of Denver had filed.

To carry water, the tunnel had to be lined throughout the western end from the intake shaft at Station 2696 + 25[5] to the apex at Station 2536 + 81 as well as through approximately 1 500 ft. of scattered seamy sections east of the apex. The remaining 12 200 ft. east of the apex is through solid granite having an average area of 100 sq. ft. (8 by 8 ft. + 50% overbreak) which would carry about 500 sec-ft. until need for the full 1 000-sec-ft. capacity necessitated enlargement or lining, or both.

According to plans contemplated, water collected by a canal system from the Fraser River, Vasquez Creek, St. Louis Creek, Ranch Creek, and tributary streams would reach the 10-ft., circular Water Tunnel through a 209-ft. vertical shaft, 12 ft. in diameter at Cross-Cut No. 3 (2 700 ft. east of West Portal). The water would be forced under pressure (gravity) over the apex, Elevation 9 244, and thence down grade to East Portal, Elevation 9 204, the hydraulic gradient being 0.004. From East Portal a short canal could deliver the water into South Boulder Creek which flows through rocky box canyons to Eldorado Springs on the edge of the plains, 15 miles northwest of Denver, and 500 ft. higher in elevation. This point is also above the head-gates of numerous irrigation ditches serving territory adjacent to the city.

VENTILATION

Air flow tests made at the Moffat Tunnel (Fig. 10) to determine the coefficients of friction of the lining preparatory to deciding upon the specifications for the blowers, furnish valuable data on the relation between flows through tunnels of railroad size and through mine tunnels. The information available on the former is fragmentary, that on the latter, voluminous; so that the rare opportunity offered by the Moffat Tunnel measurements to compare the two simultaneously as the same volume of air was forced through the 8 by 8-ft. and the 16 by 24-ft. bores, will be useful in converting the extensive experimental data on small mine airways to the solution of railway tunnel ventilation problems.

The determination of the power requirements for mechanically ventilating the tunnel, necessitated estimating both the volume of flow desired and the pressure required to circulate this quantity of air through the tunnel. In both these estimates a large number of variables are involved.

Estimating Volume of Flow.—Steam operation having been determined upon by the lessee of the tunnel, the primary purpose of ventilation would be to dilute and remove the noxious gases produced by combustion of coal in the engine. The requirements are based on diluting the carbon monoxide of the locomotive exhaust gas to a proportion that would have a negligible effect on man.

---

[5] "Construction Methods on the Moffat Tunnel," by R. H. Keays, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 66, Fig. 3.

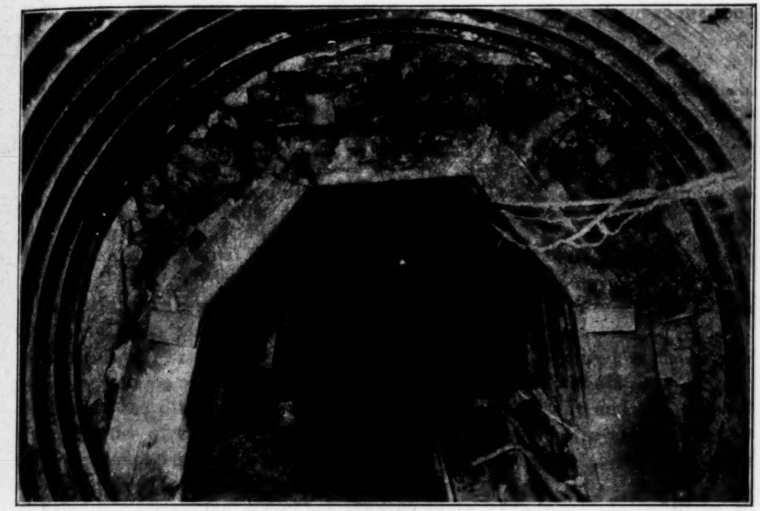


FIG. 9.—WATER TUNNEL, SHOWING WOOD TIMBER LINING BEING REPLACED BY CORRUGATED PRESSED STEEL LINER PLATES.

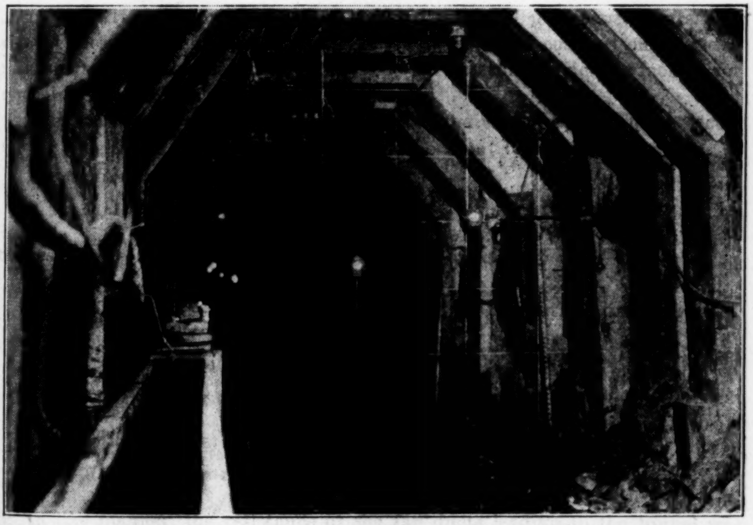


FIG. 10.—AIR-MEASURING STATION IN WATER TUNNEL—DETERMINING MEAN VELOCITY IN CROSS-SECTION.

In arriving at this quantity the following assumptions and calculations have been made:

Maximum train—locomotive and 50 cars: 2 500 tons.
Maximum tractive resistance at 24 lb. per ton on 0.9% grade: 60 000 lb.
Train speed: 10 miles per hour.
Maximum tractive power: 880 000 ft-lb. per sec.
Necessary coal consumption based on test: 0.73 lb. per sec.
Air for combustion at 25 lb. of flue gas per lb. of coal: 18.3 lb. per sec.
Air for dilution, assuming 1% ·CO in gas (based on tests), diluted to 5 parts per 10 000 (4 parts per 10 000 considered safe limit for 1 hour exposure): 330 lb. per sec.
Quantity required at 0.055 lb. per cu. ft., average weight of air at the tunnel: 360 000 cu. ft. per min.

Actually, this result is considered a maximum in that average maximum values have been assumed throughout.

Estimating Pressure Requirements.—The pressure required to cause any particular volume of air to flow through an airway, such as the Moffat Tunnel, depends on the dimensions, velocity of flow, density of air, and the physical characteristics of the airway. The latter must be handled in calculations by the use of empirical numbers which are based on experimental determinations and designated as friction factors or coefficients of friction.

Friction factors for somewhat similar types of mine airways, 40 to 60 sq. ft. in area, were available from experimental work of the U. S. Bureau of Mines, but their application to such large airways (400 sq. ft.) was extremely uncertain as was also the available evidence concerning the effect of area on friction factors, which was based on small pipe experiments. In view of these facts and the lack of preciseness of visual comparison of rough rock surfaces, it was considered necessary to determine friction factors for typical sections of the partly completed tunnel.

These tests indicated that a maximum average value of the friction factor for air weighing 0.055 lb. per cu. ft. would be about 0.000 000 0050.

Tunnel Resistance.—Using the formula,

$$R = \frac{K\,P\,L\,Q^2}{A^3}\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

the resistance of the clear tunnel was calculated as follows:

0.000 000 005 = assumed friction factor, K.
$\times$ 80 = average perimeter, P, of tunnel sections, in square feet.
$\times$ 32 250 = length of tunnel, L, in feet.
$\times$ $(360\,000)^2$ = square of estimated quantity of flow, Q, in cubic feet per minute.
$\div$ $(400)^3$ = cube of average area, A, in square feet.
= 26.16 = resistance, R, in pounds per square foot.
= 5.03 = resistance of flow, $\frac{R}{5.2}$, in inches of water.

In like manner, the resistance of the fan ducts leading to the tunnel was estimated as approximately 0.02 in. of water.

Resistance of Standing Train.—No data being available for the resistance caused by moving trains, the resistance of a standing train was first calculated from experimental data and from rational assumptions. Then the difference in results was used as a criterion in arriving at the resistance of a moving train calculated by the latter method.

With coal mine cars occupying about 20% of the total area, U. S. Bureau of Mines experiments gave an end resistance of 0.45 of the velocity pressure in the clear airway for the first car and 0.15 of the velocity pressure for the side resistance of each additional car when closely coupled. Tests on single cars of different areas indicated that the resistance varied directly as the square of the ratios of car area to airway area. Assuming that the locomotive has an effect equal to 2 cars, that the average train will comprise 50 cars, and that the cars are approximately 9 by 14 ft. in section and thus occupy about 31% of the tunnel area, the total train resistance may be calculated as:

$$\left(\frac{31}{20}\right)^2 \times [0.45 + (52 \times 0.15)] = 19.8 \text{ velocity pressures}$$

At 900 ft. per min., the velocity pressure is 0.037 in. of water for air weighing 0.055 lb. per cu. ft.; therefore, the train resistance is:

$$19.8 \times 0.037 = 0.734 \text{ in. of water.}$$

The resistance of the standing train may also be calculated by considering it made up of two components: The impact against the end, destroying $\frac{125}{400}$ of the velocity pressure, and the actual increase in resistance due to the air moving at higher velocity through the reduced section. The end resistance is then $\frac{125}{400} \times 0.037 = 0.012$ in. Assume a friction factor of 0.000 000 005; and a length of 2 000 ft., with the area reduced to 275 sq. ft., the perimeter increased to 110 ft., and the velocity increased to 1 319 ft. per min. Then the resistance through the reduced section is calculated as 1.318 in. of water. The tunnel resistance without the train is $\frac{2\,000}{32\,250} \times 5.03 = 0.312$ in., therefore the increase will be $1.318 - 0.312 = 1.006$ in. The total train resistance is then $1.006 + 0.012 = 1.018$ in., a result which is approximately 40% higher than that calculated from experimental data.

Resistance of Train Moving Against the Air Flow.—Since rational computations approximate experimental results for standing trains, it is felt that a sufficiently accurate calculation of the resistance caused by the train moving against the air flow at 10 miles per hour may be made. In this case the train velocity of 880 ft. per min. is added to the tunnel air velocity of 900 ft. per min., giving a relative velocity of 1 780 ft. per min. for the air impacting on the train, for which the velocity pressure is 0.081 in. of water. The end resistance is then $\frac{125}{400} \times 0.081 = 0.025$ in.

The velocity through the reduced section of 275 sq. ft. is 1 319 ft. per min., while the relative velocity along the train is 880 ft. per min. greater, or 2 199 ft. per min. At 1 319 ft. per min., the resistance of 2 000 ft. of this reduced section is calculated as 1.32 in., while at 2 199 ft. per min., it is 3.67 in. (See Equation (1).) Considering these values as applying in the ratios of the perimeters involved, the total side resistance would be:

$$\frac{73}{110} \times 1.32 + \frac{37}{110} \times 3.67 = 2.11 \text{ in.}$$

For this section the normal resistance without the train is 0.31 in. and the increase due to the train is 1.80 in. This, added to the end resistance of 0.025, gives a total increase in resistance due to the moving train of 1.83 in. of water. Assuming that the values thus calculated are 40% high, the train resistance is 1.31 in.

Total Resistance to Flow.—On the basis of a flow of 360 000 cu. ft. per min., the total resistance has been calculated as follows: 5.03 in. for the tunnel proper; 0.02 in. for the snowshed and fan connection; and 1.31 in. for a train moving 10 miles per hour against the flow, or a total of 6.36 in. of water.

For any other flow the combined resistance may be computed by means of the equation,

$$R = 0.49 \left(\frac{Q}{100\,000}\right)^2$$

in which, R equals resistance, in inches, of water; and Q, quantity of air, in cubic feet per minute.

Natural Draft.—Immediately after "holing through" the Water Tunnel the flow of air from west to east was measured and found to be 50 000 cu. ft. per min., corroborating the barometric readings made at the two portals.

At these points situated on the opposite sides of a mountain, pressures are rarely the same, due to differences in atmospheric conditions in the columns of air above. Barometric records have been kept at both portals since March, 1925, and these, corrected for difference in elevation, indicate that a strong natural draft from west to east predominates during the winter months, and a variable intermittent draft with frequent reversals, during the remainder of the year.

The average maximum draft west to east may be taken as about 0.3 in. of mercury (4.1 in. of water), a value that is exceeded but four or five times a year and only for relatively short periods. The maximum recorded draft has been 0.7 in. of mercury (9.5 in. of water). The average maximum draft east to west may be taken as about 0.05 in. of mercury (0.7 in. of water), a value that is rarely exceeded and but infrequently attained. The maximum recorded draft has been 0.15 in. of mercury (2.0 in. of water).

The other minor components that might be considered are the difference in weights of the air column inside the tunnel and that at the West Portal through the same elevation; and the effect of wind pressure on the portals. The former amounts to but 0.11 in. for an average difference of 40° in temperature between the inside and the outside of the tunnel, an extreme winter condition of rather short duration. Gales are frequent at both portals during the winter months, blowing against the West Portal and down over the East Portal, the latter probably producing some suction effect.

Thus, natural draft pressures tend to increase the demand upon the ventilating equipment, at least in the winter when strong adverse natural draft has to be contended with in ventilating against the heavily loaded trains moving from west to east up the 0.9 and 0.8% grades west of the apex.

Air Flow Tests to Determine Friction Factors

Air flow tests were made in March, 1927, shortly after the "holing through" of the Water Tunnel, while the main headings of the Railroad Tunnel were about 1 000 ft. apart and the enlargement operations were about 4 800 ft. apart (Fig. 11). Conditions were not ideal for experimental precision inasmuch as tunneling operations were going on day and night. However, about 20% of the observations were free from interference.

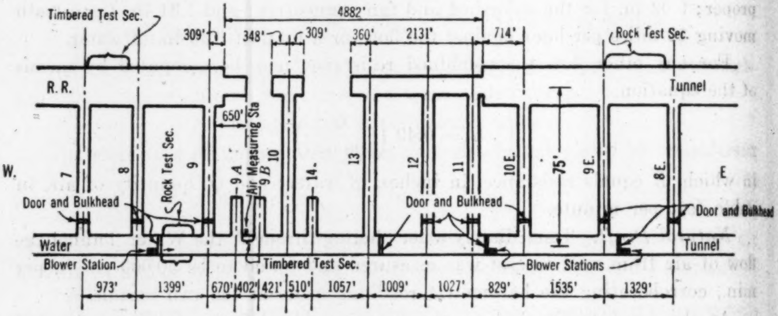

Timbered Test Sec.
4882'
309'
348'
309'
360'
2131'
714'
Rock Test Sec.
R. R.
Tunnel
650'
Air Measuring Sta
W.
7
8
Rock Test Sec.
9
9 A
9 B
10
14.
13
12
11
10E.
75'
9E.
8E.
E
Door and Bulkhead
Door and Bulkhead
Door and Bulkhead
Water
Tunnel
Blower Station
Timbered Test Sec.
Blower Stations
973'
1399'
670'
402'
421'
510'
1057'
1009'
1027'
829'
1535'
1329'

FIG. 11.—AIR FLOW FRICTION TESTS: LAYOUT OF SECTIONS, BLOWERS, AND DOORS.

Velocities were restricted by the high resistance to air flow through the 8 by 8-ft. Water Tunnel and cross-connections between the uncompleted sections of the Railroad Tunnel so that, although this arrangement insured the passage of all air through the gauging station in the Water Tunnel, it kept the maximum velocity through the Railroad Tunnel section down to about 80 ft. per min., equivalent to a flow of 35 000 cu. ft. per min.

Scope of Test Work.—Four sections (Fig. 11) were selected for tests: (1) A rock section of the Railroad Tunnel between Cross-Cuts Nos. 8-E and 9-E; (2) a timbered section of the Railroad Tunnel between Cross-Cuts Nos. 8-W and 7-W; (3) a timbered section of the Water Tunnel between Cross-Cuts Nos. 9-A and 9-B; and (4) a rock section of the Water Tunnel between Cross-Cuts Nos. 9-W and 8-W.

General Statement of Test Results.—Although individual test results varied widely, especially for short-period tests on the Railroad Tunnel sections, where very low velocities and minute pressure differences were involved, the averages of a large number of test observations seem quite reliable and check almost exactly previous estimates extrapolated from other experimental data. As a result of these tests and their comparison with available data, the friction factors shown in Table 1 were selected as applicable to such large airways.

TABLE 1.—VALUES OF FRICTION FACTOR, *K*, IN THE MOFFAT TUNNEL.

	FOR AIR WEIGHING:	
	0.055 lb. per cu. ft.*	0.075 lb. per cu. ft. (standard density).
Unlined rock	0.0000000050	0.0000000068
Timbered on 4 to 5-ft. centers	0.0000000045	0.0000000061
Timbered on 3-ft. centers	0.0000000040	0.0000000055
Timbered on 2-ft. centers	0.0000000035	0.0000000048
Timbered skin-tight	0.0000000015	0.0000000020
Smooth-lined	0.0000000010	0 0000000014

* Estimated average maximum weight of air in Moffat Tunnel.

These values are for use in the standard formula for air flow transformed from Equation (1):

$$R = \frac{K\,S\,V^2}{A \times 5.2} = \frac{K\,S\,Q^2}{A^3 \times 5.2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1a)$$

in which, K is a friction factor, which varies directly as the density of the air, and is given for a standard density of 0.075 lb. per cu. ft., unless otherwise stated; R is pressure, in inches, of water; S, rubbing surface, in square feet = perimeter times length; V, average velocity, in feet per minute; Q, quantity of flow, in cubic feet per minute; and A, the average area of cross-section, in square feet.

Test Methods.—The determination of friction factors involves the simultaneous measurement of the air flow and the difference in static pressure between the two ends of the section selected for test, together with sufficient observations of barometric pressure and temperatures for determining the weight of the air. From these data and the dimensions of the airway, values of the friction factor, K, are computed from Equation (1a).

Tests on three of the four sections required the sealing of a door on a main traffic artery, therefore, simultaneous observations were required at three widely separated points, at any one of which test observations might be, and were, interrupted for varying time intervals. Continuous observations were made at all three points for a period of several hours at a time; the results were then compared on a time basis and all those showing interference (about 80%) were discarded before computations were made. The remaining 20% were usually scattered over relatively short periods.

Under the test conditions, air measurement by calibrated anemometers was considered more practical than Pitot tube, or velocity-pressure, methods. Air flow was measured at a timber set in a heavily timbered section of the Water Tunnel just east of Cross-Cut No. 9-A. Preliminary traversing, as shown in Fig. 10, determined the relation of the mean velocity to the velocity at the center of the section and the latter only was measured during pressure-loss tests. A few center velocities, measured simultaneously by anemometer and Pitot tube, were about 2% higher by the latter method, which, considering the difference in areas involved, is ample assurance that the anemometer calibrations used were substantially correct.

Standard U. S. Bureau of Mines static tubes were set up at each end of the test section and pressures from these were communicated to a pressure gauge, near the middle of the section, through 100-ft. lengths of small ($\frac{3}{8}$-in. inside diameter) pneumatic hose connected by pure gum rubber tubing.

Wahlen differential pressure gauges were used to measure the difference in the pressures communicated through the tubings, to an accuracy of 0.0001 in. of alcohol. Temperatures in the tunnel were almost absolutely constant at any one point, but varied from 58° to 65° Fahr., in the test zone. (Rock temperatures had previously been found to range from 60° near the portals to 68° Fahr., near the center.)

Cross-sectional areas and perimeters were obtained from the graphical records for all sections except the Water Tunnel timbered section which was measured for the test.

Tunnel Conditions for Tests

Railroad Tunnel Rock Sections.—Test sections, 954 and 1 200 ft. long, through blocky granite of average roughness with some gradual area variations and ballasted floor gave an average friction factor value of 0.000 000 0070 for velocities of 57 to 83 ft. per min. (air density, 0.075 lb. per cu. ft.). Compared with previous tests on 60-sq. ft. airways having a friction factor of 0.000 000 0092, an increase of 24% corresponds to the 1:7 increase in area. The low velocities in these large airways do not appear to increase the factor materially.

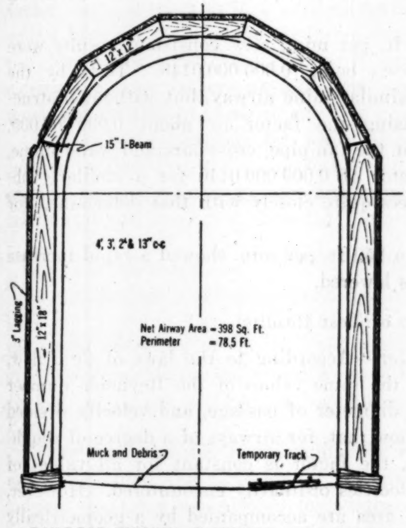
Railroad Tunnel Timbered Sections.—In the sections tested, the timbers were in almost perfect alignment, but the spacings varied irregularly as they do throughout all timbered sections of the tunnel. Expressed in terms of length of section: Three sections, totaling 12%, were timbered skin-tight; six sections, totaling 25%, on 2-ft. centers; six sections, totaling 47%, on 3-ft. centers; and two sections, totaling 16%, on 4-ft. centers. Fig. 12(*a*) shows a typical cross-section.

Four tests on a 890-ft. section, at velocities of 74 to 81 ft. per min., were for 9 to 13 min. duration and were expected to yield very consistent results. The computed results, however, were somewhat erratic and their averages of 0.000 000 0053, although roughly checked by later observations at lower velocities, may be somewhat high.

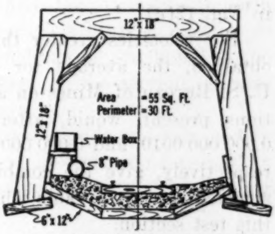
Three variables are involved in the comparison of this with results secured at high velocities in small timbered airways, namely, the effects of area, of timber spacing, and of low-velocity flow. The tests on the Railroad Tunnel rock section indicate that the latter is negligible; and mine-airways data indicate a factor, K, of 0.000 000 0070 with timbers spaced on 5.3-ft. centers, so there is an indicated reduction in K of about 32% for the effects of area and decreased spacing combined.

Water Tunnel Timbered Section.—In this section, between Cross-Cuts Nos. 9-A and 9-B, the 12-in. faces of the timbers were in good alignment with but few irregularities and small offsets. The timbers were spaced 18 and 24 in., center to center, in alternate blocks, about one-third the length involved being timbered on 18-in. centers and two-thirds on 24-in. centers.

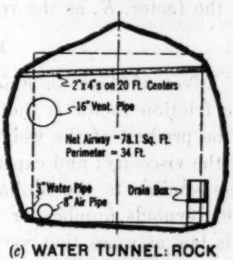
The floor was of loose muck with the track ties exposed throughout and with the bottom segments exposed for about one-third the length on one side. A water flume, approximately 1.0 by 1.1 ft., occupied part of one side of the section, resting on the floor at the east end, with the space beneath filled with pipes and blocking at the west end. Areas were taken inside the upper braces with no allowance for the spaces between these braces and the caps, since tests have shown that no appreciable flow occurs through such small openings. The shape of the section is shown in Fig. 12(*b*); and the type of surfaces in Fig. 10.


12"x12"
15" I-Beam
4', 3', 2'& 13" c-c
5" Lagging
12"x18"
Net Airway Area = 398 Sq. Ft.
Perimeter = 78.5 Ft.
Muck and Debris
Temporary Track

(*a*) RAILROAD TUNNEL TIMBERED TEST SECTION


12"x18"
Area = 55 Sq. Ft.
Perimeter = 30 Ft.
12"x18"
Water Box
8" Pipe
6"x12"

(*b*) WATER TUNNEL: TIMBERED TEST SECTION


2"x4"s on 20 Ft. Centers
16" Vent. Pipe
Net Airway = 78.1 Sq. Ft.
Perimeter = 34 Ft.
3" Water Pipe
8" Air Pipe
Drain Box

(*c*) WATER TUNNEL: ROCK TEST SECTION

FIG. 12.—AIR FLOW FRICTION TESTS: TYPICAL CROSS-SECTIONS OF TUNNELS USED FOR MEASUREMENTS.

Since all observations were under sight control, 2-min. tests yielded quite consistent results even on different, but overlapping, test sections and in spite of the interference resulting from working on a small main traffic artery. Six tests on six different sections, 310 to 320 ft. in length, gave an average factor, *K*, of 0.000 000 0053 at velocities of 600 to 700 ft. per min. At velocities of 340 to 400 ft. per min., four tests on two sections gave an average factor of 0.000 000 0051.

Later tests at low velocities, with a small amount of timber storage in the test section, gave factors as high as 0.000 000 0073 at a velocity of 171 ft. per min. At 264 ft. per min., the factor was 0.000 000 0057 and, making some allowance for the added obstructions present, it seems that, for this section, a break in the law of flow occurs at a velocity of about 250 ft. per min.

An average value of *K* of 0.000 000 0052 is indicated for this section for velocities greater than 250 ft. per min. For a quite similar section timbered on 5.3-ft. centers, a factor of 0.000 000 0080 has been determined in mine airways. There is thus an indicated decrease of about 35% as between 5.3 and 1.8-ft. spacing in airways of this size.

Water Tunnel Rock Section.—In this section, between Cross-Cuts Nos. 8-W and 9-W, the rock surfaces were more irregular and the area gradations were more abrupt and of greater extent than average conditions in railroad tunnel sections; and not only was the floor much rougher, but the section had a number of obstructions, including a fan pipe, 14 in. in diameter, with flanged joints, a water-flume and blocking, a 3-in. water pipe, an 8-in. air pipe, and 4-in. cross-bars on 20-ft. centers near the roof. A representative section is shown in Fig. 12(*c*).

For velocities greater than 250 ft. per min., very consistent results were obtained, the average for eight tests being 0.000 000 0148. Tests by the U. S. Bureau of Mines on a quite similar mine airway, but without obstructions present, would, after increasing the factor by about 0.000 000 005, 0.000 000 0010, and 0.000 000 0010 for the fan-pipe, cross-bars and water-flume, respectively, give the combined factor of 0.000 000 0140 for a similarly obstructed airway, a value which agrees quite closely with that determined for this test section.

A few tests at velocities less than 250 ft. per min. showed a rapid increase in the factor, *K*, as the velocity was lowered.

Evaluation of Test Results

Effect of Area on Friction Factor.—According to the laws of fluid flow, the friction factor is the same for the same values of the Reynolds number —the product of the weight of air, diameter of passage, and velocity divided by the viscosity; and experiments show that, for airways of a degree of roughness similar to the Moffat Tunnel, the factor is constant for all values of the Reynolds number, or for all velocities ordinarily encountered. However, this law assumes that variations in area are accompanied by a geometrically similar variation in surfaces, and does not apply exactly where variations in area are accompanied by a constant degree of roughness of surfaces.

In estimating factors for Moffat Tunnel sections, approximately 400 sq. ft. in area, from test data on 40 to 60-sq. ft. mine airways of the same absolute degree of roughness, the difference between absolute similarity and geometrical similarity had to be estimated. From rather scanty data the accompanying decrease in factor was estimated at about 25% and the tests on the Railroad Tunnel rock section exactly confirm this result if the assumption is made that the determined factors are not influenced by low-velocity flow. Test conditions did not permit determining the effect of the latter for the large railroad sections, but the results obtained indicate that it was either quite small or negligible. Its effect, if present, would be to increase the factor and to increase the percentage of decrease of factor over that determined. The uncertainty is thus on the safe side.

Effect of Low-Velocity Flow on Friction Factor.—Tests on the rock and timbered sections in the Moffat Water Tunnel show a rapidly increasing factor for velocities of less than about 250 ft. per min. Tests on the Railroad

Tunnel sections could only be made for a very limited range of low velocities, but both the determined value of the factors and their close agreement, for the small velocity range of 57 to 83 ft. per min. in the rock section, indicate that, for these large sections, the effect of low-velocity flow on the factor is either quite small or negligible.

Effect of Timber Spacing on Friction Factor.—The tests in the timbered sections of the Railroad Tunnel corroborated determinations made in the Water Tunnel which showed that, for the close spacing of 5 ft. or less, the friction factor decreased with the spacing of the sets. Inasmuch as laboratory tests tend to indicate that spacing timbers far apart would decrease the friction factor, there appears to be a critical spacing at which the air begins to form eddies between the timbers, and this spacing apparently bears a relation to the airway area. The results in the Railroad Tunnel can be attributed to the closeness of the timbers as compared with the area of the section, a ratio much smaller than that which exists in mine airways.

Although the purposes for which these tests were made did not justify further expenditure, the comparative data obtained indicate the possibilities of more extensive investigations and were found helpful in estimating the air friction of the completed Railroad Tunnel.

VENTILATING PLANT

A ventilating plant corresponding to these friction factor findings was built and is being used by the Denver and Salt Lake Railway Company for the steam operation of the railroad tunnel. A 500-h.p. fan with a capacity of 350 000 cu. ft. per min. and a stand-by 750-h.p. fan, with a capacity of 450 000 cu. ft. per min., are housed in a concrete building forming the East Portal.

They are connected to the tunnel by ducts with hinged steel dampers by means of which the direction of the air currents can be reversed, so that induced draft can be provided for westbound traffic and forced draft for eastbound. A vertically lifting canvas door on a steel frame closes the East Portal during periods of ventilation and is raised automatically when a train approaches. Air-current velocities vary from about 10 to 15 miles per hour.

Consulting Engineers.—The Board of Consulting Engineers retained by the Moffat Tunnel Commission was comprised of the late David W. Brunton, M. A. I. M. M. E., *Chairman,* J. Waldo Smith, Hon. M. Am. Soc. C. E., and L. D. Blauvelt and J. Vipond Davies, Members, Am. Soc. C. E.

This Board rendered valuable service during the construction of the project. When unexpected difficulties demanded utmost co-operation and resourcefulness it suggested, among other expedients, the use of the steel wall-plates. Its recommendation that the Water Tunnel be lined west of the apex is being gradually accomplished.

ECONOMIC VALUE OF THE TUNNEL

The project involved $15 470 000 outstanding bonds. The first issue amounted to $6 720 000 at 5½%; the second, $2 500 000 at 5¼%; the third, $3 500 000 at 5¼%; and the fourth, $2 750 000 at 5 per cent. Real estate of the Moffat Tunnel Improvement District, of which Denver represents 89%, is security for these bonds, and a total cost, including earnings of camps and interest during construction, of $18 000 000, in round figures, must be of pronounced economic value to justify this investment.

The purpose of the tunnel was threefold:

1.—To eliminate about 30 miles of 4% grade over Rollins Pass, lowering the maximum altitude from 11 660 ft. to 9 240 ft., which is below the snow areas, reducing the ruling grade to 2.0% and the maximum curvature to 12°, and eliminating 10 800° of curvature; all of which was important to the life of the Denver and Salt Lake Railroad.

2.—To place Denver on a through transcontinental rail route, saving 173 miles between Denver and Salt Lake City, by either building the 41-mile Dotsero Cut-Off, connecting the Denver and Salt Lake Railroad with the Denver and Rio Grande Western Railroad, or extending the Denver and Salt Lake Railroad from its present terminus, Craig, to Salt Lake City, Utah; and at the same time to assure adequate transportation facilities for a vast area in Northwestern Colorado rich in timber, coal, oil, oil shale, live stock, and agricultural and mining resources.

3.—To provide a conduit for the transportation of water from the Upper Colorado River on the western slope to the dry eastern slope around Denver, for municipal and agricultural purposes.

Progress toward accomplishing these results may be noted:

(1) The physical railroad improvement has been accomplished.
(2) Negotiations for the construction and use of the proposed Dotsero Cut-Off are under way with both the Denver and Rio Grande Western and Denver and Salt Lake Railroads as probable users. Applications are before the Interstate Commerce Commission.
(3) The City of Denver has entered into a contract for the use of the Water Tunnel as a carrier for its Fraser River water. It is proposed under this contract that the City pay $1.00 per year rental, maintain the water bore, and eventually assume the cost of actual lining as part of its water-supply development program.

The indirect returns from these proposed uses of the tunnel should be increasingly great, obscuring the difficulties of construction and constituting another chapter in the story of the Moffat Tunnel.

APPENDIX I

MOFFAT TUNNEL—COST DATA

ITEMIZED COST OF ROCK EXCAVATION IN 8 BY 8-FOOT TUNNEL AND HEADINGS. (Average overbreak beyond 64 sq. ft. = 50%; cost based on yardage within 8 by 8-ft. pay line = 171 580 cu. yd.; these costs do not include plant cost, camp, or interest charges, or overhead beyond the superintendent.)

Item No.	Labor.	Cost per cubic yard.	Item No.	Supplies.	Cost per cubic yard.
(a)	Drilling and blasting	$1.564	(a)	Drill steel	$0.229
(b)	Hand-mucking (clean up)	0.454	(b)	Air hose	0.029
(c)	Machine mucking	1.051	(c)	Air pipe	0.205
(d)	Scaling	0.150	(d)	Water hose	0.014
(e)	Hauling in tunnel	0.955	(e)	Water pipe	0.075
(f)	Disposal	0.380	(f)	Ventilating pipe	0.129
(g)	Track-laying	0.660	(g)	Electric lights and wiring	0.392
(h)	Pipe lines	0.224	(h)	Trolley	0.037
(i)	Ventilation	0.221	(i)	Dynamite	1.497
(j)	Lights and wiring	0.259	(j)	Tamping rags	0.003
(k)	Power house	0.127	(k)	Fuse	0.020
(l)	Sharpening steel	0.336	(l)	Blasting wire	0.081
(m)	Repairs, mucking machines	0.153	(m)	Exploders	0.178
(n)	General repairs	0.162	(n)	Oil, grease, and waste	0.067
(o)	Drainage	0.357	(o)	Storage battery supplies and repairs	0.011
(p)	Superintendence	0.250	(p)	Locomotive repairs	0.077
(q)	Shifters	0.407	(q)	Car repairs	0.052
(r)	Drill repairs	0.059	(r)	Tool repairs	0.033
(s)	Car repairs	0.070	(s)	Track in tunnel and dump	0.300
(t)	Trolley	0.023	(t)	Drill parts	0.251
	Miscellaneous, including "jumbos", drayage, trimming, bonus, and special work	0.248	(u)	Mucker repairs	0.210
			(v)	Power	0.650
			(w)	Miscellaneous, including drayage	0.057
			(x)	Steel sharpening	0.105
			(z)	Tools, replacements	0.042
			(a-b)	Drainage	0.074
			(a-a)	Pumps	0.015
	Total labor cost per cubic yard	$8.110		Total supplies per cubic yard	$4.777

Total cost per cubic yard, labor and supplies $12.89

Total cost per cubic yard, labor and supplies, in solid rock $10.41

APPENDIX II

MOFFAT TUNNEL—COST DATA

ITEMIZED COST OF ROCK EXCAVATION IN ENLARGEMENT FROM 8 BY 8-FOOT TO 16 BY 24-FOOT STANDARD, SINGLE-TRACK RAILROAD TUNNEL.

(Average overbreak = 18%; cost based on pay line yardage = 433 293 cu. yd.; these costs do not include plant cost, camp, or interest charges, or overhead beyond the superintendent.)

Item No.	Labor.	Cost per cubic yard.	Item No.	Supplies.	Cost per cubic yard.
(*a*)	Drilling and blasting	$0.996	(*a*)	Drill steel	$0.140
(*b*)	Hand-mucking (clean up)	0.684	(*b*)	Air hose	0.025
(*c*)	Machine mucking	0.481	(*c*)	Air pipe	0.039
(*d*)	Scaling	0.041	(*d*)	Water hose	0.009
(*e*)	Hauling in tunnel	0.597	(*e*)	Water pipe	0.001
(*f*)	Disposal	0.301	(*f*)	Ventilating pipe	0.001
(*g*)	Track in tunnel	0.115	(*g*)	Lights and wiring	0.093
(*h*)	Pipe lines	0.071	(*h*)	Trolley	0.018
(*i*)	Ventilation	0.001	(*i*)	Dynamite	0.433
(*j*)	Lights and wiring	0.081	(*j*)	Tamping rags	0.001
(*k*)	Power house	0.067	(*k*)	Fuse	0.035
(*l*)	Sharpening steel	0.165	(*l*)	Blasting wire	0.006
(*m*)	Air-shovel repairs	0.035	(*m*)	Exploders	0.045
(*n*)	General repairs	0.125	(*n*)	Oil, grease, and waste	0.043
(*o*)	Drainage	0.017	(*o*)	Storage battery repairs	0.002
(*p*)	Superintendence	0.099	(*p*)	Locomotive repairs	0.058
(*q*)	Shift bosses	0.304	(*q*)	Car repairs	0.029
(*r*)	Drill repairs	0.030	(*r*)	Tool repairs	0.024
(*s*)	Car repairs	0.049	(*s*)	Track repairs	0.098
(*t*)	Trolley	0.013	(*t*)	Drill parts and repairs	0.152
(*u*)	Trimming	0.097	(*u*)	Shovel repairs	0.050
	Miscellaneous, including "jumbos," drayage, special work, etc.	0.206	(*v*)	Power	0.328
			(*w*)	Miscellaneous	0.084
			(*x*)	Steel sharpening	0.064
			(*y*)	Jumbo set	0.056
			(*z*)	Tool replacements	0.032
			(*a-a*)	Pump repairs	0.003
			(*a-b*)	Drainage	0.003
			(*b-b*)	Refuge bays	0.001
Total labor cost per cubic yard		$4.575	Total supplies per cubic yard		$1.823

Total cost per cubic yard, labor and supplies .... $6.398 = $6.40

Total cost per cubic yard, labor and supplies, in solid rock $4.80

APPENDIX III

MOFFAT TUNNEL—COST DATA

COST OF TIMBER LINING

(In dollars per 1 000 ft. B. M. in place; for 1 000 ft. B. M. per linear foot of tunnel, see attached typical timbering drawings; standard 5-segment, 12 by 12-in. set = 1 050 ft. B. M.; spark protection = 10-ft. wide canopy of 1 by 6-in. California Redwood nailed to permanent segmental arch timbers.)

Item.	Labor.	Material.	Total.
Oregon fir in railroad tunnel	57·11	52·67	109·78
Native Lodgepole pine in railroad tunnel	121·84	56·56	178·40
Timber in water tunnel	88·53	27·79	116·32
Cost per linear foot of tunnel	1·01	0·85	1·86

APPENDIX IV

MOFFAT TUNNEL—COST DATA

AVERAGE COST OF STEEL LINING IN 16 BY 24-FOOT RAILROAD TUNNEL.

Item.	Labor.	Material.	Total.
Arch	$ 99·71	$252·88	$352·59
Pair of posts	151·19	353·15	504·34
Invert set	31·74	84·17	115·91
Complete set	$282·64	$690·20	$972·84
Cost per linear foot of tunnel sets on about 3-ft. centers	$112·34	$234·74	$347·08

Cost per linear foot of concrete lining (inverts, side-walls and arches), 8·5 cu. yd. at $31.74 = $270.00

DISCUSSION

J. Vipond Davies,[6] M. Am. Soc. C. E. (by letter).—In order to give a complete story of the work, this valuable paper must be read in connection with the previous one by R. H. Keays, M. Am. Soc. C. E.[1] Certain aspects which had influence and bearing on the final result, are considered in the following discussion.

In 1913, the Tunnel Commission of the City and County of Denver, by its Board of Engineers, Messrs. Brunton, Davies, and Finch, after considering carefully a geological report prepared for the Commission, reported on the project, recommending the present general location and profile of the tunnel. Further, under pre-war conditions, estimates of the probable cost of the tunnel construction were prepared. At that time the question of water supply was not looked upon as important, but to meet the requirements of the law it was contemplated that pipe conduits might be laid in the floor of the tunnel itself. It was also contemplated that the railroad tunnel would be equipped and operated by electric power. The geological report indicated no likelihood of encountering anything but sound hard rock, although it might be of varying texture and character.

In the preparation of the Moffat Tunnel Law, no reconsideration was given to the excessive advances in cost under post-war conditions, and only an arbitrary addition was made, as stated in the New Act, in the sum of $2 300 000 over that provided in the earlier law, which had been declared by the Courts of Colorado to be unconstitutional.

The Engineering Board and Staff consequently came into the problem of construction of a new project with hands already tied, on a definite location and elevation, with an estimated cost declared in the text of the law, and with a real problem for water supply to be included. The Commission had power to equip and electrify the tunnel and approaches for operation. In earlier years this electrification had been desired and planned by the Railroad Company, and, it was anticipated, would be installed in the completed tunnel.

Previous to the preparation of plans for the tunnel, as now constructed, the Board visited the site with the State Geologist, who indicated that the glacial valley erosion, on the west slope, might be deep, so that if a change were made in the location by a few hundred feet in a southerly direction, it would ensure the entire western half of the tunnel being in solid rock throughout. As is now known, that view was only partly correct, for, although the rock existed continuously, its physical structure was most deficient. The doubt on the part of the Geologist, was that the glacial valley would be so deep as to intersect the tunnel section in the branch of Buck Creek nearest the Western Portal. There was no doubt expressed as to the quality of bedrock where found.

With rock of more or less uniform character there appeared no reason why progress in driving should not have been equal from both ends; consequently, the grade of 0.8% originally contemplated was increased to 0.9%, with the apex of the eastern and western gradients as nearly as practicable at the middle point of the tunnel. This increased grade would be no detriment if the railroad were to be operated with electric power, particularly as the approach grades at both ends are 2% up to the tunnel portals. The eastern gradient of 0.3% was maintained for proper drainage. The work of construction was in active progress with those established gradients, when the Railroad Company determined that it would not electrify for operation, but would use steam power, for which it considered the 0.8% gradient to be vital, and artificial ventilation to be an absolute essential.

---

[6] Cons. Engr.; Pres., Jacobs & Davies, Inc., New York, N. Y.

In view of the fact that the rock generally from the Eastern Portal to the ridge of the Rocky Mountains was hard and practically dry, it was not thought that the driving of the eastern end over the apex for a longer distance would be of material consequence. From this change arose one of the serious difficulties which developed later and has been recorded.

Mr. Betts refers to the absence of diamond drill borings. It is open to question whether such borings, if made, would have altered the situation. It appears certain that the pre-conception of the Geologist as to the depth of the glacial valleys on the western slope was probably under-estimated. An attempt was made, before finally determining the axis location, to dig pits from the surface in order to locate the rock floor below the west branch of Buck Creek, only to find glacial moraine of boulders and gravel, bearing heavy water. The rock in the tunnel on the western side of the Rockies, to outward observance, is sound native material, but has been rotted, probably by water infiltration on imperfectly fused material. It seriously lacks supporting strength. Even in the small section of the Water Tunnel, it would not carry its own loads without close timber support.

Further, the moraines are a vast reservoir of water from melted snow, as that is practically the only source of water to creeks or on the soil, from higher elevations. As Mr. Betts states, while only a few thousand feet of the tunnel was ever anticipated to require artificial support, the final result proved that every bit of the western half needed both temporary and permanent support. Further, the heavy water encountered in the tunnel driven from the East Portal, was tapped practically under the summit of the Continental Divide. This clearly shows that the fissures in the rock through which the water flowed came from the great Ranch Creek Moraine on the west slope of the mountains.

As long as the railroad was expected to be electrified, there was little concern as to the ventilation of the 6¼ miles of tunnel through the mountain, particularly as the usually existing higher barometric pressure at the western end, added to the difference in actual elevation of the portals, made it certain that there would be adequate current of air to provide purity for passengers and workmen, as well as for cattle, which constitute an important element in the traffic eastward. When, however, it was finally determined that the railroad would be operated by steam power, the problem of ventilation became of vital importance, necessitating determination of those factors relative to which little or no authoritative information was available, particularly as to the frictional resistance to the movement of air in such a tunnel. The Engineering Profession is deeply indebted for these investigations to the U. S. Bureau of Mines, its engineers, and to Mr. Betts. The present method of operation and ventilation can be considered efficient only until the time is reached when traffic over this route increases so as to require electrification.

The general administration of the work was excellent throughout, both in respect to the Commission and its officers, as well as on the part of the Contractors and their staff. The high elevation and remoteness of the camps were sources of anxiety from the first. The original specifications, prepared in the hope of obtaining straight unit price bids, were greatly elaborated so as to obligate any contractor to install and maintain the most complete possible equipment, sanitary facilities, and general camp hygiene. Notwithstanding the fact that the provisions of the contract and administration of the work were completely changed, those provisions as outlined were carried out to the fullest limit, to the great advantage of the work.

R. H. Keays,[7] M. Am. Soc. C. E. (by letter).—This valuable contribution supplements the paper previously submitted by the writer,[1] especially relating to events after 1925. The very valuable description of the ventilation experiments and the ventilation plant move one to attempt to supplement this feature and make a few general remarks on the ventilation of long railroad tunnels.

Besides the system described by the author, which is dependent upon direct air pressure or suction, there is the well-known Sacchardo System of induced ventilation. In this system the mass of air in the tunnel is pushed forward by the pressure caused by numerous jets of high-velocity air, introduced at one of the portals of the tunnel. These air jets are arranged in an annular ring and point into the tunnel parallel to the walls.

The Sacchardo System requires a ventilating fan to work against a much higher pressure than the direct displacement system. Because there is a rather definite and low limit to the pressure to be developed by a fan, and because the longer tunnels naturally require greater pressures, it is very difficult to apply the system to the longest tunnels.

An important objection to its application is that in the Sacchardo System the current of air is not reversible unless one has a plant at each end. The great advantage of the system, however, is that both portals of the tunnel are open at all times, so that the ventilation system will always function. With the direct-pressure system, there can be no ventilation when the closing of the curtain door is obstructed by a passing or standing train.

In any system of ventilation the question also arises as to whether to blow with or against the movement of the train. In the system as described by Mr. Betts, the ventilation is against the train. Ventilating with the movement of the train, however, is somewhat safer in that the smoke from the locomotive travels ahead rather than back among the passengers and crew. To blow with the movement of the train, however, requires much greater volume and velocity of air, unless the speed of the train is kept very slow.

---

[7] Chf. Engr., Ulen & Co., Athens, Greece.

In the case of the Moffat Tunnel, it is impracticable to rely on the Sacchardo System installed at either end. At the east end, the air pressures would have to be made high enough to overcome the large and almost continuous difference in the barometric pressure of the atmosphere between the two portals, while, if installed at the West Portal, the blowing would have to be in the same direction as the heavy traffic. A plant installed at the West Portal would also cost much more than one at the East Portal because the electric power, presumably used for operation, comes from a generating plant on Boulder Creek, which is some distance from the East Portal. A power line over the range would be difficult to maintain and a line through the tunnel itself would be very expensive.

The Sacchardo System, as the writer understands it, was first applied on one of the Apennine Tunnels, in Italy, and was so successful that later it was used on all other mechanically ventilated tunnels in Europe, with the single exception of the Simplon Tunnel. In each case the blowing was against the heavy traffic. In the case of the Simplon Tunnel, which is single track and more than 12 miles long, it can be imagined that some very high pressures would have been necessary with the Sacchardo System to induce sufficient draft. The only recourse, therefore, was the direct-pressure system.

The writer has had the pleasure of passing through most of the long tunnels of Europe and wishes to note here that there is apparently no longer any ventilation problem because all the railways have been electrified. Trains pass through at high speed, and one is hardly aware of the existence of the tunnel.

A. A. EREMIN,[8] ASSOC. M. AM. SOC. C. E. (by letter).—This paper, together with that entitled "Construction Methods of the Moffat Tunnel", by R. H. Keays, M. Am. Soc. C. E.,[1] is of much value to the profession; it describes many interesting engineering and economical features of tunneling. Moreover, in his reference to the late David H. Moffat, the author calls attention to the fine public spirit of one engineer who worked with untiring activity for the welfare of a city.

The author has stated that observations on the lining of the tunnel have proved the statement made by the Board of Consulting Engineers that if weak rock is held securely it has a tendency to reach a constant distributed bearing, thus insuring permanent safety. This statement is in agreement with the theory of pressure on deeply overlaid tunnels as developed by Kommerell.[9]

When a tunnel is excavated in a soil of uniform cohesion an arch effect occurs over the bore, and the pressure on the lining will be expressed by the weight of the area of the parabola, $A\ B\ C$, in Fig. 13 (*a*). If a side pressure occurs, the loading diagram will be expressed by the shaded area in Fig. 13 (*b*). The curve, $A'\ C'\ B'$, is a parabola or, for practical purposes, it may be considered as an ellipse. Since the height, H, in Fig. 13, is a function of the cohesive properties of the ground, a comparatively uniform bearing pressure will occur in the arch roof of the tunnel unless it is complicated by some physical or chemical action in the soil.

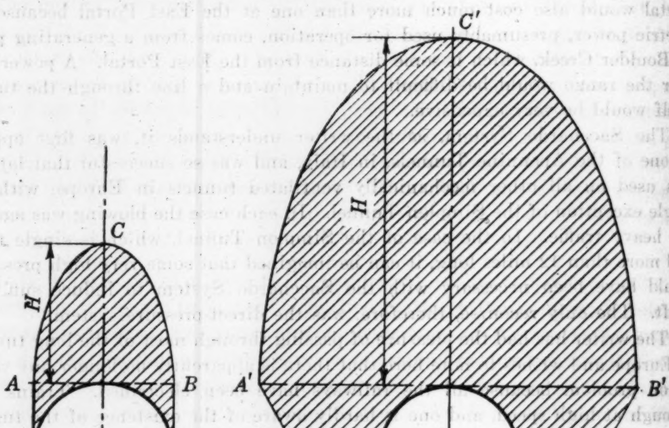
---

[8] Asst. Designing Engr., Bridge Dept., State Highway Comm., Sacramento, Calif.

[9] Kommerell, "Statische Berechnung von Tunnelmauerwerk"; see, also "Tunneling," by Eugene Lauchli, 1915, p. 104.

In discussing timber lining, the author refers to the previous statement[10] by Mr. Keays that "the decision to try to hold this heavy ground permanently with timber only was a bold one."

From Fig. 13, it is evident that the pressure on the tunnel lining varies with the size of the tunnel and the arching effect of the ground and that it does not depend on the depth of the overlying mass.



(a) (b)

FIG. 13.—DIAGRAMS SHOWING EARTH PRESSURE ON TUNNEL LININGS.

The most common cause of failure in the timber of a segmental arch rib is due to shear at the joints.[11] Many factors can be blamed for this local development of shear, namely, poor workmanship, local and uneven distribution of swelling in the ground, soft material, unsymmetrical stratification of the ground in relation to the center line of the tunnel, and an insufficient number of segments used in the arch ring.

In actual practice it is very seldom that the soil penetrated by a tunnel is of uniform formation. Usually, it is cut by seams and fissures and shattered by faults. A geological survey is expensive and, therefore, in order to have a greater factor of safety, a structural material with better resistance to shear and flexural stresses than timber is usually selected for the lining of deep tunnels. A serious disadvantage of timber lining is the difficulty of preserving it from decay. High temperature, moisture, and chemicals dissolved in the ground are very injurious.

---

[10] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 95.

[11] *Engineering News-Record*, December, 1927, p. 1039.

Useful information concerning soil pressure may be obtained from an observation of timber lining left for some time to support the ground before enclosing it in concrete for the permanent lining.

Information on tunneling is scarce in technical literature. Therefore, this paper is especially important to the Engineering Profession.

Clifford Allen Betts, M. Am. Soc. C. E. (by letter).—The valuable discussions have brought out some important points. Mr. Davies' reference to the limitations imposed on the Engineering Staff by legislative action and, later, by legal contracts, illustrates the interdependence of economic feasibility, sound engineering construction, and adequate provisions for repayment of cost, all of which are essential to the ultimate success of projects in general and of this one in particular.

The merits of electrification, especially where a heavy volume of traffic is involved, are indicated in Mr. Keays' outline of European practice, which he has had an unusual opportunity to study. Electrification of the Moffat Tunnel had been taken for granted up to the time the lessee exercised the option of steam operation. The capacity of the tunnel has been estimated at forty trains daily. This use is contingent upon utilization of the Dotsero Cut-Off for which the way was cleared in 1930 by the United States Court decisions validating the lease with the Denver and Salt Lake Railroad Company and by the Interstate Commerce Commission's approval of the acquisition of the Denver and Salt Lake Railway by the Denver and Rio Grande Western Railroad Company.

The effect of steam operation on this partly timbered, 6-mile tunnel will be watched with interest in order to determine whether the chemical action of hot flue gases and the subsequent forced ventilation drafts accelerate deterioration. In this connection, the statement by Mr. Eremin that "a serious disadvantage of timber lining is the difficulty of preserving it from decay", is apropos and explains the recommendation that a concrete-lining program be carried out as maintenance replacements of timber become necessary. This would permit deriving the maximum use of the investment in treated timber lining, would defer large interest charges on concrete, and would provide an opportunity to study the unusually severe ground pressures prior to concreting the remainder of the bore.

There is a wide range in physical and chemical properties of ground. The Moffat Tunnel contained numerous formations from blocky, solid rock unaffected by weathering to a squeezing and swelling fault zone material that produced upward pressures in the tunnel floor, as well as top and side pressures. This type of ground (which might be added to the two illustrated by Mr. Eremin in Fig. 13) is difficult to hold because the flowing pressures find the weakness of the lining whether it be by shear at the joints or by bending of segmental arch ribs or posts. In shortening timber arch members to approach a circle the stability of the arch is endangered, but if steel members are used, the circular arch provides most effective resistance to such pressures. This is being borne out in the Water-Tunnel lining.

An excellent opportunity to substantiate Mr. Eremin's statement that "pressure on the tunnel lining varies with the size of the tunnel and the arching effect of the ground", occurs in two tunnels with which the writer is connected. A 3.55-mile water tunnel, of the horseshoe type, 16 ft. 7 in. in diameter, is being driven at approximately right angles to a 4.15-mile tunnel, 9 ft. 3 in. in diameter, with only 300 ft. separating the inlet portal of one from the outlet of the other. Where similar lava and tuff are encountered in each, the smaller tunnel stands without support, while the larger bore requires 10 by 10-in. fir timbers on 5-ft. centers to hold the ground until the concrete is placed. The difference in pressure between the two tunnels is very noticeable. Advantage can be taken of this fact in determining the economic balance between cost of lining and the excavation costs in designing tunnel sizes.

Recent developments on the Colorado River emphasize the strategic location of the Moffat Water Tunnel for diversion of a water supply for Denver, Colo., from the head-waters of that stream. Due to the rapidly increasing value of water in the arid West and the natural limitations of the watersheds available for serving Denver by gravity, this Water Tunnel has great potential value.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1772

RAINFALL CHARACTERISTICS AND THEIR RELATION TO SOILS AND RUN-OFF[1]

BY C. S. JARVIS,[2] M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. GEORGE S. BLISS, BREHON B. SOMERVELL, JESSE W. SHUMAN, CLINTON L. BOGERT, R. W. POWELL, GLEN N. COX, C. R. PETTIS, GEORGE C. DANFORTH, AND C. S. JARVIS

SYNOPSIS

Investigators of projects involving either flood control or the development of water resources frankly recognize the need for additional records of primary rainfall observations, and also for a fuller use of those already collected and published.

Ever-increasing and voluminous reports appear from time to time dealing with meteorology, hydrography, and soil moisture; with the development, surveys, and classification of soils; with erosion and its control; with the varying yields of water-sheds; and with the volume of floods, and their probable frequency of occurrence. All these data must be rigorously condensed in some manner before they can be used to the best advantage.

The purpose of this paper is to present summarized information regarding precipitation and its occurrence in various countries and latitudes; its relation to soils and run-off; the vegetative, topographic, and physiographic features of the water-sheds; and the resultant influence on designs of drainage structures and channels.

FORM OF DATA

Precipitation records for the principal stations throughout the world have been assembled in compact form in Table 1 (Appendix I), with both the

---

[1] Revision and extension of a thesis submitted for the degree of Ph. D., at American University, Washington, D. C., 1927; written with co-operation of U. S. Bureau of Public Roads. Published in January, 1930, *Proceedings*.

[2] Prin. Hydr. Engr., U. S. Engr. Office, War Dept., Washington, D. C.

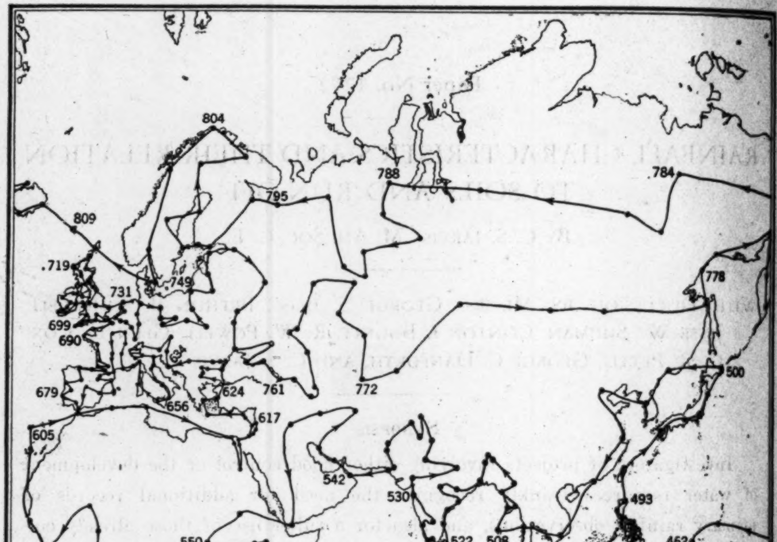

804
788
784
795
809
.719
749
731
699
690
778
679
624
761
772
500
656
617
605
542
530
493
550
522
508
462
482
556
475
596
586
472
467
567

FIG. 1.—KEY MAP OF WORLD SHOWING SEQUENCE OF STATION NUMBERS FOR PRECIPITATION RECORDS—EASTERN HEMISPHERE.

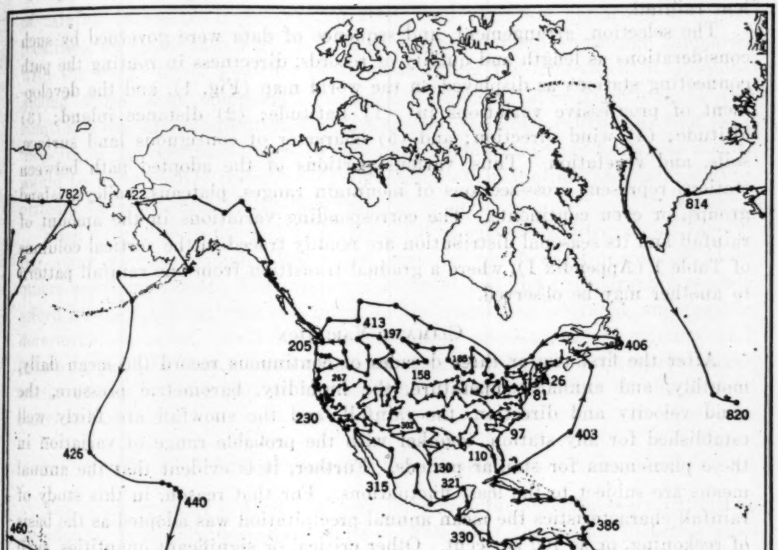

782
422
814
413
197
205
406
188
158
267
26
81
820
230
302
97
403
110
426
130
321
315
440
386
460
330
378
445
360
453
365

FIG. 1.—KEY MAP OF WORLD SHOWING SEQUENCE OF STATION NUMBERS FOR PRECIPITATION RECORDS—WESTERN HEMISPHERE AND PACIFIC OCEAN.

average habits and the observed range of variation clearly represented. The references from which these data were obtained are given in Appendix II. Wherever snow or hail is included in the observations, it is reduced to equivalent rainfall.

The selection, arrangement, and sequence of data were governed by such considerations as length and quality of records, directness in routing the path connecting stations as displayed on the world map (Fig. 1), and the development of progressive variations in: (1) Latitude; (2) distance inland; (3) altitude; (4) wind direction; and (5) character of contiguous land surface, soils, and vegetation. Thus, various portions of the adopted path between stations represent cross-sections of mountain ranges, plateaus, valleys, island groups, or even continents. The corresponding variations in the amount of rainfall and its seasonal distribution are readily traced in the vertical columns of Table 1 (Appendix I), where a gradual transition from one rainfall pattern to another may be observed.

CLIMATIC VARIATION

After the first two or three decades of continuous record the mean daily, monthly, and annual temperature, the humidity, barometric pressure, the wind velocity and direction, the rainfall, and the snowfall are fairly well established for any station, together with the probable range of variation in these phenomena for similar periods. Further, it is evident that the annual means are subject to the least fluctuations. For that reason, in this study of rainfall characteristics the mean annual precipitation was adopted as the basis of reckoning, or as 100 per cent. Other critical or significant quantities, such as the maximum observed monthly precipitation and the mean for the corresponding month, also the maximum for 24 hours, have been reduced to percentages on the same basis for convenient comparison.

Inspection of the ten percentage columns in Table 1 (Appendix I), discloses more than a mere semblance of harmony. Beside the somewhat definite patterns traced by various groups, there are orderly progressions and trends from one group to another, with variations in latitude, altitude, temperature, distance inland, prevailing wind direction, and other conditions that influence relative humidity. Whether the tentative classifications be based on one or all of those factors, the great mass of divergent data conforms more readily to a few simple patterns of behavior than the primary observations would indicate.

CLIMATIC STABILITY

No marked progressive changes of climate or of rainfall habits have been noted during several centuries of record; therefore, by determining what part of the pattern is represented by fragmentary, detached, and short-period records, a definite clue or forecast is provided concerning meteorological phenomena to be expected during longer intervals. The observed periodic changes usually complete their fluctuations within forty years, or thereabouts, according to authoritative opinion among meteorologists; but some stations require only twenty, whereas others need more than seventy, years for that purpose.

DETERMINATE LIMITS

While it has long been recognized that there may be somewhat definite limits of rainfall intensity, frequency, duration, and variation applicable to given localities, an appreciation of the limited scope of human knowledge compared with the broad fields awaiting exploration deters the expression of opinions among those most competent to speak. Probably approximations to the limiting rates of precipitation for various periods and localities, as well as a measure of the resultant excessive run-off or surface drainage known as floods, would be more clearly indicated by arranging, co-ordinating, and interpreting primary observed data more effectively.

Such natural phenomena may be regarded as successive integrations involving many variable complex influences, some of which are interrelated while others are independent. Fortunately, meteorological and hydrographic records afford means for occasional evaluation of the quantities delivered, and further determinations may result from such simple processes as proportion and interpolation. In this manner solutions of practical problems reported herewith were undertaken during the assembling and tabulation of the accompanying basic data.

NEW VIEWS OF OLD PROBLEMS

The need for flood control has come into prominence with every extensive inundation of highly developed valleys. Following the recent disasters sustained in widely separated districts of the United States, where flood-protection systems had been installed and immunity from serious damage had been supposedly secured, the public conscience has been awakened to the National responsibility in this vital problem and its solution. The resulting legislative enactments of 1928 which aimed to combine the most modern methods with the most positive means for effective flood control to promote the general welfare, are as noteworthy as were the catastrophes which prompted the legislation.

In carrying out flood-control programs recently authorized by Congress, engineers need to utilize every available means for extending their knowledge of rainfall characteristics, run-off rates corresponding thereto, and related phenomena in the vicinity of selected projects. Additional meteorological and hydrographic stations will be required to supplement those already established for the collection of primary data; but it is equally important to devise methods for assembling, co-ordinating, and utilizing the detached and apparently unrelated fragments of such records. If this can be accomplished, the effective periods of observation will be thereby extended far beyond their present limits, and the designs for retarding basins and waterways may embody proper consideration for both economy and safety.

If the source of supply and also the most frequented routing or paths of storms are known for a given area, and some fragmentary data are available to disclose any part of the rainfall pattern, then the information in Table 1 (Appendix I), should furnish something of an index for completing the other parts of the record by comparison with available data pertaining to stations similarly situated and drawing on the same supply. Apparently, the characteristic percentage relations hold fairly consistently in such cases, regardless of the variation in rainfall traceable to the physical configuration of the land surface, except in arid regions.

Outstanding examples to illustrate that principle are furnished by the percentage columns for New England stations, others in Germany, and such adjacent but quantitatively varying stations, as Oviedo and Bilboa, in Spain (Nos. 683 and 684); Ben Nevis and Fort William, in Scotland, if adjusted to the same period of record (Nos. 717 and 718); and Tiflis and Lenkoran, in Trans-Caucasia (Nos. 762 and 763).

Both length and reliability of records, as well as completeness of detailed information and its representative character for either groups, areas, or the range of their habits, were sought among stations included in Table 1 (Appendix I).

INTERRELATION BETWEEN METEOROLOGICAL AND HYDROLOGICAL PHENOMENA

An adequate understanding of the cause-and-effect relation between rainfall and run-off must likewise include the range of conditions which may influence the rate, sequence, and continuity of the various natural processes involved in the rain-producing cycle. Such conditions may include solar radiation, atmospheric circulation and convection, evaporation, expansion, cooling, condensation, deposition, percolation, stream flow, and the accretions or losses sustained en route.

Often arrested or interrupted processes are observed in the meteorological cycle, such as extended cloudy periods without rainfall, or the more advanced stage in which a downpour actually begins in the upper atmosphere, but is dissipated in the lower strata before reaching the earth's surface, producing the familiar cloud-drapery appearance above desert regions. Likewise, the detention and regulation effected by the soil and vegetation during moderate showers may interrupt the run-off and reduce it to a negligible quantity. Under ideal conditions practically all the rainfall percolates into the voids of the soil, and emerges therefrom at a gradually decreasing rate until new accessions of moisture are available.

Maximum river stages usually result from the coincidence of either intense rates or prolonged periods of precipitation or of thawing, together with low absorptive, interceptive, and retentive powers of the soil and its cover.

WATER-SHED CHARACTERISTICS

Types, structure, color, and stages of development of soils and the varieties of plant growth sustained thereon, provide a supplemental record awaiting interpretation as to rainfall and run-off habits, whether they be irregular and violent or steady, gentle, and dependable. Thus, the 143 sq. miles of drainage area along Queen Creek, in Central Arizona, with an average annual precipitation of 15 in., produced a high flood volume of 9 000 cu. ft. per sec., but maintained an average flow of only 15 cu. ft. per sec. throughout the year, representing a yield of 10% from the water-shed; by comparison, an equal area along Cedar Creek, in Northwestern Washington, with an average annual rainfall of 120 in., attained an observed maximum flow of 3 600 sec-ft., a minimum of 294 cu. ft. per sec., and an average discharge of 1 089 cu. ft. per sec., a yield representing nearly 90% of the total observed precipitation. Both the soils and the vegetation that abound in those two regions emphasize the contrast between the steady, gentle, dependable rainfall of the Pacific Northwest as opposed to the sudden, irregular, brief, torrential downpours that are so characteristic of desert regions. The soft, mossy banks, the tender foliage, and the sodded slopes are identified with precipitation which develops almost imperceptibly from fog or mist; while the spike foliage, thorny stems, and other resistant equipment of desert growth reflect the struggle for existence.

There are known physical limitations as to the moisture capacity of the atmosphere for given temperatures and pressures, and there may be just as definite limits for barometric pressure gradients and the wind velocities induced thereby; likewise for storm movements and areas affected, rates of evaporation and condensation, heating and cooling, also freezing and thawing, for designated seasons and localities.

LIMITATIONS GOVERNING ATMOSPHERIC MOISTURE

The high transparency of clear, dry air allows the transmission of nearly 90% of the normal solar radiation under most favorable conditions. The losses due to reflection, absorption, and conduction are all increased by the presence of foreign substances, including water vapor, so that the radiant energy reaching the earth is reduced accordingly. After condensation has proceeded far enough to form dense cloud masses, practically the entire solar radiation is intercepted.

An average measure of the sun's radiant energy that reaches the outer atmosphere may be expressed as 2 calories per sq. cm. per min., or $\frac{1}{5}$ h.p. per sq. ft. of normal surface continuously during sunlight hours. If transmitted to the earth without loss, this energy would be capable of evaporating a 1-in. depth of water at 72° Fahr. during 12 hours near the equator, or from two-thirds to one-half this quantity in mid-latitudes. However, the actual toll of energy taken by the atmosphere during average fair weather conditions is nearly 50%; and during cloudy periods it becomes 90%, or more.

Taking into account the varying opportunities for evaporation as determined by the prevailing temperatures, the inclination of the sun's rays, the

TABLE 2.—RECORD OF EVAPORATION RATES.

(All depths in inches.)

Station	Years of record.	MAXIMUM.			MEAN.	ANNUAL.			Remarks.
		Day.	Mean month.	Month.	April to September.	Minimum.	Mean.	Maximum.	
Gardiner, Me	3	......	5.0	5.8	22.8	..	26	..	....
Boston, Mass	15	......	6.0	7.4	28.6	31	39	48	....
Rochester, N. Y.	25	......	4.7	6.9	....	26	31	39	....
Washington, D. C	3	......	...	7.0	32.2	..	..	..	....
Chapel Hill, N. C	4	......	5.3	7.5	....	..	40	..	....
Charleston, S. C	20	......	5.7	8.6	....	30	47	66	Reservoir.
Charleston, S. C	19	......	10.4	....	....	..	88	..	Land pan.
Birmingham, Ala	..	......	7.5	....	38.3	..	51	..	....
California, Ohio	..	......	7.2	....	35.5	..	46	..	....
Cleveland, Ohio	6	......	...	....	24.6	..	..	..	....
Detroit, Mich	4	......	...	....	32.1	..	..	..	....
Thunder Bay, Mich	4	......	...	....	30.2	..	..	..	....
Milwaukee, Wis	5	......	...	....	26.9	..	..	..	....
Lake of the Woods, Minn	3	......	3.5	....	20.7	..	..	..	....
Columbia, Mo	2	......	...	7.5	31.2	..	50	..	....
Crowley, La	7	......	6.2	....	32.3	..	..	..	....
Laredo, Tex.	1	......	...	13.7	68.4	..	..	..	....
San Antonio, Tex	11	......	9.6	....	46.0	..	..	..	....
Austin, Tex	2	......	...	11.0	47.3	..	68	..	....
Amarillo, Tex	11	......	10.4	....	52.7	..	..	..	....
Big Springs, Tex	3	......	12.7	....	59.0	..	..	..	....
Dalhart, Tex	10	......	10.6	....	55.4	..	..	..	....
Lawton, Okla	3	......	9.0	....	45.6	..	..	..	....
Woodward, Okla	4	......	10.8	....	49.5	..	..	..	....
Garden City, Kans	10	......	10.9	....	52.8	..	..	..	....
Hays, Kans	11	......	10.0	....	47.9	..	..	..	....
North Platte, Nebr	11	......	9.1	....	43.6	..	54±	..	....
Lincoln, Nebr	11	......	....	....	34.8	..	..	..	....
Newell, S. Dak.	16	......	8.1	....	36.6	..	..	..	....
Edgeley, N. Dak	11	......	6.4	....	29.6	..	..	..	....
Mandan, N. Dak	5	......	7.4	....	33.3	..	..	..	....
University, N. Dak	16	......	5.7	7.1	26.2	..	..	..	....
Huntley, Mont	13	......	7.5	....	32.8	..	..	..	....
Moccasin, Mont	9	......	7.1	....	32.8	..	..	..	....
Archer, Wyo	5	......	7.9	....	36.3	..	..	..	....
Laramie, Wyo	6	......	....	....	39.4	..	..	..	....
Fort Collins, Colo	16	......	....	....	29.3	....	....	....	....
Akron, Colo	10	......	9.1	....	42.7	....	....	....	....
Grand Valley, Colo	1	......	....	....	36.5	....	....	....	....
Rocky Ford, Colo	1	......	....	....	61.6	....	....	....	....
Tucumcari, N. Mex	6	......	10.9	13.8	55.4	....	....	....	....
Santa Fe, N. Mex	2	......	....	11.9	52.5	....	....	....	....
Elephant Butte, N. Mex	7	......	14.3	....	....	90	102	110	....
Elephant Butte, N. Mex	2	......	....	16.9	75.8	....	....	....	....
Elephant Butte, N. Mex	..	......	13.5	....	....	....	87	....	....
Carlsbad, N. Mex	4	......	....	12.4	69.4	....	107	....	In city.
Carlsbad, N. Mex	4	......	....	11.1	....	....	94	....	In alfalfa field.
Lake Avalon, N. Mex	10	......	10.0	12.9	....	66	73	83	....
Las Cruces, N. Mex	3	......	....	....	40.1	....	....	....	....
Holbrook, Ariz	5	......	....	8.6	47.0	....	....	....	....
Roosevelt Reservoir, Ariz	7	......	13.4	14.8	67.0	82	87	95	....
Granite Reef, Ariz	1	......	....	12.7	....	....	98	....	Tank floating in river.
Granite Reef, Ariz	1	......	....	14.3	....	....	115	....	Tank on ground
Mesa, Ariz	2	......	12.2	....	56.2	....	91	....	....
Tucson, Ariz	3	......	....	....	54.2	....	....	....	....
Yuma Desert, Ariz	3	......	17.5	....	....	114	123	136	....
Yuma, Ariz	9	......	10.6	....	55.1	....	96	....	....
Yuma, Ariz	5	......	10.4	....	....	70	76	88	Alfalfa field.
Nephi, Utah	10	......	9.5	....	44.6	....	....	....	....
Spanish Fork, Utah	5	......	9.6	....	38.8	....	....	....	....
Fort Douglas, Utah	3	......	....	....	30.7	....	....	....	....
Aberdeen, Idaho	6	......	9.6	....	42.1	....	....	....	....
Arrowrock Idaho	8	......	....	10.2	42.0	....	....	....	....
Deer Flat, Idaho	8	......	8.0	....	38.1	....	96	....	....
Minidoka, Idaho	..	......	....	15.0	....	....	96	....	....
Walla Walla, Wash	2	......	....	9.5	38.0	....	....	....	....

TABLE 2.—(*Continued*).

(All depths in inches.)

Station.	Years of record.	MAXIMUM.			MEAN.	ANNUAL.			Remarks.
		Day.	Mean month.	Month.	April to September.	Minimum.	Mean.	Maximum.	
North Yakima, Wash.	..	......	....	10.7	....	....	68	....	....
Lake Kachess, Wash.	8	......	....	6.1	23±	....	33	...	....
Klamath, Ore.	4	......	8.5	9.2	....	....	51	....	....
Hermiston, Ore.	6	.....	8.5	11.1	36.6	....	....	....	....
Hermiston, Ore.	..	......	8.4	....	....	....	44	....	....
Moro, Ore.	7	......	9.3	....	41.0	....	....	....	....
Fallon, Nev.	16	......	10.7	....	50.9	61	64	73	....
Fallon, Nev.	..	......	....	9.9	....	....	54	....	Tank floating in canal.
Lake Tahoe, Calif.	7	......	4.9	6.5	....	20.5	30	40.1	....
Lake Tahoe, Calif.	8	.....	5.9	....	....	....	40	....	....
Biggs, Calif.	4	......	9.6	....	....	....	44	....	....
Sweetwater, Calif.	7	......	....	9.0	....	49	54	59	....
Cuyamaca, Calif.	9	......	....	....	....	45	57	85	....
Fresno, Calif.	..	......	....	....	....	....	66	....	....
Los Angeles, Calif.	..	......	....	....	....	....	37	....	....
San Diego, Calif.	..	.....	....	....	....	....	38	....	....
Mammoth, Calif.	..	......	....	18.0	....	....	126	....	....
Owens Lake, Calif.	..	......	....	....	....	....	61	....	....
Christiansted, Virgin Islands.	8	......	8.2	....	45.2	74	78	84	....
Assuan, Egypt.	3	......	....	....	....	....	107	....	....
Bombay, India.	..	......	....	....	....	....	96	....	....
Van Wijks Vlei, South Africa.	14	......	....	....	....	....	67	....	....
Kimberley, South Africa.	8	......	....	....	....	79	89	93	Vaal River.
Kimberley, South Africa.	8	......	....	....	....	....	55	....	Deep reservoir.
Bloemfontein, South Africa.	..	......	....	....	....	66	....	82	Exposed to wind.
Bloemfontein, South Africa.	..	......	....	....	....	48	....	58	Sheltered from wind.
Johannesburg, South Africa.	..	......	....	....	....	....	75	....	....
Capetown, South Africa.	..	......	....	....	....	....	79	....	....
Port Elizabeth, South Africa.	..	......	....	....	....	32	....	46	....
Alice Springs, Australia.	24	0.88	12.5	15.5	....	34	96	108	....
Adelaide, Australia.	47	0.62	9.0	11.2	....	47	54	61	....
Eucla, Australia.	6	0.52	7.0	8.3	....	53	58	61	....
Brisbane, Australia.	3	0.48	....	8.2	....	49	51	55	....
Blackall, Australia.	4	0.70	11.5	13.0	....	77	83	92	....
Boulla, Australia.	3	0.91	....	16.7	....	....	111	125	....
Murrumbidgee, Australia.	5	...	....	....	....	29	37	45	....

soil moisture content, the degree of cloudiness, the vegetation and foliage, the wind movements, and the atmospheric humidity, it is estimated that, on an average, the equivalent of 0.1-in. depth of water over the entire surface of the globe becomes vapor each day. The upper limit of evaporation rates attainable from free water surfaces may equal 1 in. daily. Illustrative of local variations, the recorded annual evaporation from a tank on the ground at Indio, in the Imperial Valley, California, was 119 in.; from a tank 10 ft. above the ground, 200 in.; from the saline waters of the Salton Sea itself, 72 in.; from a tank 2 ft. above the surface of Salton Sea, 106 in.; from a tank 40 ft. above the water, 140 in.; from a tank on the desert, 165 in.; and from one that was elevated 40 ft. above the desert, 195 in.

Representative evaporation rates for several important stations are shown in Table 2. Main references to this table may be found in Appendix III.

Evaporation Compared with Precipitation

Comparison of the corresponding columns in Table 1 (Appendix I) and Table 2 indicates the normal relationship between evaporation and precipitation phenomena, especially as regards maximum observed rates. The evaporation process is more or less continuous, while rainfall is usually periodic; the one is governed by atmospheric circulation and temperatures affecting surfaces of known or readily ascertainable areas, while the other depends on condensation of cloud volumes at considerable heights, in air strata which may be moving in different or opposite directions from those below.

TABLE 3.—Moisture Capacity of Atmosphere at Various Temperatures.*

Temperature, in degrees Fahrenheit.	Weight, in Pounds, of 1 000 Cu. Ft. of Air.		Moisture Percentages by:		Equivalent Depth of Water, in Inches.	
	Dry.	Saturated.	Weight.	Volume.	Per 1 000 ft. of air column.	Summation for successive strata.
—45	95	95	0.01	0.01	0.001	0.001
—40	94	94	0.01	0.01	0.001	0.002
—35	93	93	0.01	0.02	0.002	0.004
—30	92	92	0.02	0.03	0.003	0.007
—25	91	91	0.02	0.04	0.004	0.011
—20	90	90	0.03	0.05	0.005	0.016
—15	89	89	0.04	0.06	0.006	0.022
—10	88	88	0.05	0.08	0.008	0.030
— 5	87	87	0.06	0.10	0.010	0.04
0	86	86	0.08	0.12	0.013	0.05
5	85	85	0.1	0.2	0.02	0.07
10	84	84	0.1	0.2	0.02	0.09
15	83	83	0.2	0.3	0.03	0.12
20	83	83	0.2	0.3	0.03	0.15
25	82	82	0.3	0.4	0.04	0.19
30	81	81	0.3	0.5	0.05	0.24
35	80	80	0.4	0.6	0.06	0.30
40	79	79	0.5	0.8	0.08	0.38
45	78	78	0.6	1.0	0.09	0.47
50	78	78	0.7	1.2	0.11	0.58
55	77	77	0.9	1.4	0.14	0.72
60	76	76	1.1	1.8	0.16	0.88
65	75	75	1.3	2.1	0.19	1.07
70	75	74	1.5	2.5	0.22	1.29
75	74	73	1.9	3.0	0.26	1.55
80	73	72	2.2	3.6	0.30	1.85
85	72	71	2.6	4.2	0.35	2.20
90	72	71	3.0	4.8	0.41	2.61
95	71	70	3.5	5.6	0.47	3.08
100	71	69	4.1	6.6	0.54	3.62
105	70	68	4.8	7.7	0.63	4.25
110	70	68	5.5	8.9	0.72	4.97
115	69	67	6.4	10.3	0.82	5.79
120	69	66	7.4	11.8	0.94	6.73
125	68	65	8.6	13.9	1.07	7.80

*These average values were computed for normal pressure at sea level and were based mainly on Smithsonian Physical Tables. The last column quantities represent moisture content subject to precipitation.

Except for those few stations of extremely high or low rainfall, there are corresponding values to be found for both monthly and yearly means and maxima; but the range of variation from normal is not so wide for evaporation, representing receipts of atmospheric moisture, as for precipitation, representing expenditures. The brief mid-day showers of the tropics exemplify the expenditure of an entire day's accumulation of atmospheric moisture in less than an hour. The maximum rates recorded in Table 1 (Appendix I) for 15-min. periods occasionally approach or exceed the greatest observed daily evaporation from free water surfaces. Many of the inequalities as well as the similarities displayed by those two phenomena are traceable to limitations governing atmospheric moisture capacity.

Under the extreme conditions immediately preceding a storm, with the temperature as high as 110° Fahr. at sea level, for example, and the air column closely approaching saturation, the moisture subject to precipitation is nearly 2.8 in. in the first mile above the base; 1.3 in. in the second mile; 0.6 in. in the third; and 0.3 in. above the 3-mile level, according to Table 3. For such a condition, a 5-in. depth of rainfall (Table 3) may be the attainable limit without renewal of the moisture supply by wind movements. Whether a multiple or only a fraction of that amount may be registered as precipitation is beyond present human knowledge unless the records of past performance may serve as a guide.

TABLE 4.—COMPARISONS OF COMPUTATIONS.*

Station.	Season.	MOISTURE CAPACITY OF SATURATED ATMOSPHERE ACCORDING TO FIELD OBSERVATIONS.					Corresponding Values from Table 3.†
		Observed Data.				Computed Data.	
		Latitude, in degrees.	Temperature, in Degrees Fahrenheit.		Height explored, in miles.	Moisture Capacity at Saturation.	Portion subject to precipitation, in inches.
			Maximum.	Minimum.		Depth, in inches.	
McMurdo Sound..	Summer	78 S.	18	— 49	3.7	0.20	0.14
Arctic Sea	Summer	77 N.	41	— 62	6.8	0.80	0.40
Pavlovsk..........	Summer	59 N.	52	— 54	8.7	1.16	0.64
	Winter	59 N.	18	— 69	7.4	0.28	0.14
England..........	Summer	52 N.	61	— 60	8.7	1.40	0.92
	Winter	52 N.	37	— 71	8.7	0.60	0.33
Canada	Summer	43 N.	68	— 74	6.8	1.84	1.20
	Winter	43 N.	30	— 80	9.3	0.60	0.24
St. Louis, Mo.....	Summer	38.6 N.	77	— 60	9.3	2.28	1.67
	Winter	38 6 N.	34	— 80	9.3	0.68	0.29
Atlantic Ocean...	Summer	20.7 N.	77	— 83	8.7	2.60	1.67
Victoria Nyanza..	Summer	0	73	—108	12.5	2.92	1.41

*In the Dictionary of Applied Physical Science, Vol. III, p. 61, is a chart displaying the data observed and derived by several carefully conducted series of investigations.

†As the portion of the moisture content at saturation which is subject to precipitation varies from nearly 50 to 75%, this is nearly the correct proportion of the total, shown in the preceding column.

Field observations recorded in Table 4 for atmospheric moisture capacities at widely separated stations have furnished a practical and satisfactory test for Table 3, whether applied to frigid, temperate, or torrid regions. In most instances both summer and winter conditions were observed.

Not all the moisture content of either the air or the soil is subject to removal by natural processes; a considerable percentage remains as residue continuously. The moisture that may be removed from the air column by precipitation during a storm is probably not more than one-half or three-fourths of the total capacity at saturation.

For various soil types the readily removable moisture may range from more than 90% for sand to less than 20% for heavy clay. The effective storage capacity and also the contribution to stream flow during dry seasons are reduced accordingly, and thus the opportunity for evaporation may be limited by what the soil will relinquish when the heat units and atmospheric capacity for moisture are both available.

It is evident that the normal rates of evaporation for various latitudes or for different climatological zones are such as would surcharge the atmosphere within a period of a month, or less, under any conceivable conditions, provided there is ample opportunity for evaporation from either free water surfaces or wet soil, or for transpiration from luxuriant vegetation and foliage. The surcharged atmosphere obtains relief either by increased temperature and moisture capacity, or by precipitation.

If proper allowance be made for the required interval of lag between the processes of evaporation and precipitation, together with the effects produced by progressive seasonal and local variations of temperature and relative humidity of the atmosphere, some of the characteristic rainfall habits may be explained, or, perhaps, they may be forecast. Likewise, the influence of barriers, whether they be mountains or masses of dense, cold atmosphere obstructing the path of moisture-laden air currents, can be partly evaluated.

The most dependable controls for such methods of analysis seem to be the long-period records kept by weather bureaus, supplemented by a consideration of the fundamental physical laws involved. Where only meager records are available, it seems advisable to attempt the expansion of fragmentary data in terms of what appears to be the attainable limiting conditions against which provision should be made in the design of waterways for a given locality. Table 1 (Appendix I) and Table 2 were compiled to furnish something of a background for interpreting the more detailed data which may be assembled for specific problems, in the same manner as previously used for hydrographic investigations.[3]

It appears that precipitation may vary either directly or inversely with evaporation, depending on local conditions which determine whether the moisture supply is steady or intermittent; whether the source is near-by or remote; whether the routes to be traversed by the winds are clear or obstructed, both approaching and beyond the station in question; and whether the latent energy represented by the vapor content of the atmospheric column is capable of producing a violent, or only a moderate, disturbance upon release by condensation.

---

[3] "Flood Flow Characteristics," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 985.

The Energy of Vaporization and Condensation.—The energy required to change 1 lb. of ice at 32° Fahr. to water amounts to 144 B. t. u., equivalent to the work of elevating that same weight 144 by 778 ft., or to a height of 21.2 miles. The same quantity of heat would raise 144 lb. of water from 60° Fahr. to 61°; or 1 lb. of water from 32 to nearly 176 degrees.

Evaporation of 1 lb. of water under average atmospheric pressure at sea level requires the expenditure of 1 069 B. t. u. at a temperature of 40°; 1 052 B. t. u. at 70°; or 1 036 B. t. u. at 100° Fahr. Thus, at ordinary temperatures and pressures, such as prevail while most of the process takes place, it requires approximately seven times as much energy to vaporize a given weight of water as to melt the same amount of ice; or if converted into units of work, it represents an expenditure that would elevate the same weight to a height of nearly 150 miles.

Because the relative weights of air and water vapor at equal temperatures are as 1.0 : 0.62, it is apparent that the presence of 3.0% by weight, or 4.8% by volume, of moisture to produce saturation at 90° Fahr., as shown by Table 3, will reduce the weight of the entire volume by 1.8% as long as condensation is delayed. A condition of marked instability, therefore, may result if opportunities for evaporation are alternately favorable and unfavorable. At 70° Fahr., 1%, and, at 110° Fahr., 3.4%, of variation in weight and consequent pressure may be due to the vapor content.

The condensation of 1 in. of rain releases the equivalent of 2 h.p.-hours of energy per sq. ft. of surface, or nearly 87 000 h.p.-hours per acre. If applied exclusively toward heating the first mile in height of air column above sea level, the resultant increase of temperature would be 30° Fahr., or more, depending on the density and humidity; and this thermal change would cause an expansion ranging from 6 to more than 10 per cent. The resulting buoyancy would be of the same order as that possessed by ice or logs floating in water.

Although it is improbable that all the latent energy released during a storm is devoted solely to heating a portion of the atmospheric column, it is definitely known that a large part of it induces energetic convection currents which continue until equilibrium is finally established or approached through ascent, expansion, cooling, intermixing, and condensation. Much of the heat is introduced into the soil by the absorption of rain water and thereby promotes bacterial activity, the release of soil fertility, and the rapid growth of vegetation, which is so noticeable after a storm, especially one that is prolonged and of only moderate intensity.

TIME ELEMENT INVOLVED IN EVAPORATION AND CONDENSATION

Pure, dry air may transmit solar radiation with losses of less than 10% when the sun is near the zenith. Other highly transparent substances of equivalent mass, such as a 34-ft. depth of clear, placid water, or a 14-ft. thickness of the best grade of glass, behave similarly. The amount of energy delivered from the sun at such times is capable of evaporating nearly $\frac{1}{12}$ in. in depth of water every hour, or of melting $\frac{1}{2}$ in. in thickness of ice, or of raising the temperature of either a 6-in. layer of dry soil, or of a 1 000-ft. stratum in the lower atmosphere fully 25° Fahr. Thus, a few consecutive hours of intense sunshine would render the tropics and even the temperate zones unsuitable for most forms of organic life, were it not for the moderating influences of moisture and wind.

Fully 12 hours of very bright sunshine, or the combined action of persistent wind and moderate temperature, are required to evaporate 1 in. in depth from a free water surface. Such a rate is rarely exceeded except in shallow pools or in places exposed to vigorous wind movements. Condensation of the same quantity ordinarily occupies several hours, but in extreme cases precipitation, the culmination of the process, occurs within a few minutes in greater amount. Virtually, it is bulk delivery as opposed to the deliberate complexities of manufacturing.

The intimate knowledge and accurate interpretation of meteorological conditions which were so largely stimulated by the development of aviation, have gradually taken definite form, and one of the new departures, that is, official weather forecasts a full week in advance, is well within if not beyond the experimental stage. Thereby the major storm movements and the general tendencies on large areas are foretold with nearly the same degree of accuracy as the more detailed local weather forecasts have attained for small areas.

SOILS AND THEIR MOISTURE CAPACITY

The products of rock-weathering ultimately become soils through changes wrought by micro-organisms with the assistance of climatic and biological forces. Prior to the bio-chemical alterations the decomposed or disintegrated material has not developed its character sufficiently to permit classification under soil types; it is regarded, therefore, as in the embryonic stage. As the biological activity extends only a few feet below the surface, and chemical, hydrolytic, thermal, and other disturbing influences are most pronounced in the same zone, the upper layer to a depth of 10 ft., or less, claims most of the attention of soil investigators.

Recent Conceptions Concerning Soils.—Since 1900 the foundation of a radically new conception of soil science has been established in Russia and developed in Europe and America, until it now occupies a major position commanding a broad field. Old ideas regarding the dominant influence of the parent ledge were abandoned after it was demonstrated that a given type of soil may develop from any of the most prominent varieties of rock magma, provided the environment is favorable. From this view has evolved the notion that soil is not a mere inert residue or product of decay, but a complex entity, a creature of growth and culture with varying expressions of vitality. Its habits seem to reflect the influences of the temperature, rainfall, and drainage which governed its development, as well as that of lower forms of organic life which are so intimately associated with fertility and fruitfulness.

Carrying the simile still further, it has been aptly observed that the coarser granular particles constitute the framework or skeleton and determine the stability of the body structure. Again, the colloids combine the functions of tendons, muscles, linings, digestive system, and surplus tissue, for here the stores of essential minerals and other plant food lie quiescent until requisitioned by the circulatory system to supply the needs of vegetation.

There is a well-defined respiratory process by which oxygen is introduced and carbon dioxide removed; complexions of red, gray, brown, or black, disclose the degree of oxidation and hydration, and also the presence of iron or other coloring pigments that have survived the process of leaching. Here, as in organisms, impaired drainage facilities may produce a pronounced toxic condition. The living roots readily resist the soil digestive process, but the dead wood soon succumbs to the yeasts, moulds, and bacteria swarming within the colloids. Finally, soil degenerates under improper treatment and responds so gratefully to regular care and understanding that it seems like an expression of life and health, rather than of decay.[4]

SOIL CHARACTERISTICS

A comparison of the desirable qualities or characteristics of soils brings forth striking contrasts. For example, the presence of considerable organic matter is objectionable in soil for structural foundation and highway purposes; and it is either beneficial or vitally essential in the case of soil for agricultural use and stream regulation. For example, the presence of considerable organic matter is objectionable in the first case, and either beneficial or vitally essential in the second. On the other hand, high percentages of coarse, inert, granular fragments of rock are desirable for sustaining either structures or traffic and also for regulating run-off, inasmuch as stability and ready drainage facilities are among the prime requisites. These considerations, however, do not govern in farming operations; there, the sieve-like structure of coarse, gravelly, or sandy soil has become almost a synonym for sterility or lean production.

Where the only available sub-grade material is a fertile soil, it requires some sort of treatment to render it suitable for sustaining either structures or traffic. Dragging, rolling, or other manipulation while the soil is wet may afford improved stability and resistance to accessions of moisture. Both the results and the methods used are contrary to recommended practice in agriculture or in the maintenance of water-sheds to regulate run-off, for in such activities it is considered that the permeability, the normal capillary structure, and the organic life of the soil should be maintained and not destroyed. The readiness with which an overturned bucket of water is absorbed in a meadow or lawn, in contrast with its behavior on a well-trodden pathway where saturation is long delayed and where there is never danger of miring, illustrates the difference between productive soils under a cover of vegetation and the same materials after puddling and compression, erosion and baking, have rendered them sterile and inert.

---

[4] "Soils and Civilization," by Milton Whitney, Chf. of the Bureau of Soils, U. S. Dept. of Agriculture, p. 25, "The Soil as a Living Thing."

It is an unusually close-textured soil that cannot absorb and temporarily hold 1 in. of rainfall for each foot in depth of the soil deposit, provided no previous saturation, freezing, baking, or mechanical compaction has occurred to seal the inlets. In the deep soils traversed by irrigation systems of the West, it is customary to sustain a loss of 10 to 20 in. of water daily over the entire wetted perimeter of earth canals, regularly throughout the season. In humid regions only unusual soil structure, such as occurs in coarse sandy or glacial deposits, would be capable of comparable behavior; yet in lesser degree all soils of close texture may function similarly, especially if covered with dense vegetation.

For ordinary rates of rainfall the toll taken by vegetation and soil to prevent or regulate run-off is much greater than the capacity of detention basins usually designed and provided by works of Man. The flood volume recorded in Lake Champlain within a week following the unusually severe storms of early November, 1927, was not more than one-half the actual precipitation as estimated by several independent investigators, including official observers. For restricted areas near the head-waters, nearly 90% appeared as run-off, but this may have been further depleted on its way to distant outlets.

Soil Classifications.—Soils are sometimes classified on a chemical basis, as silicate, carbonate, sulfate, or ferruginous deposits, each with suitable subdivisions. Another system utilizes the manner of deposition, as residual, transported, alluvial, colluvial, aeolian, or glacial, with subdivisions according to parent sources, as quartzite, granite, dolomite, or basalt.

The Russian scientist, Glinka, has proposed a general classification based on the moisture conditions under which the types were developed, whether excessive, optimum, average, moderate, or insufficient; with sub-classes such as tundra, podsol, chernozem, prairie, gray desert, laterite, and red loam, each identified by color, position, texture, composition, and details regarding the formation generally.

The United States Department of Agriculture, through the Bureau of Soils, has developed what appears to be the most logical and comprehensive classification yet devised, incorporating the outstanding features of all other important systems, each in its natural position. The primary category is based on the presence or absence of calcium carbonate accumulations; the second on the degree of oxidation, stage of leaching, formation of sesquioxides, and the differences in organic content. The third category identifies the soil type, such as laterite, podsol, or chernozem; the fourth has to do with the "soil profile", that is whether or not the characteristic changes have developed at the various "horizons", or depths below the ground surface, from top-soil to sub-soil.

Further subdivision deals with geographical regions and soil provinces, as Piedmont, Upland Valley, or Coastal; the locality where a given type was first surveyed and classified, as the Upshur, Miami, or Volusian series; and, finally, the texture and composition are described, as gravelly loam, light silty clay, or heavy gumbo.

The results already achieved in the United States as well as abroad justify the belief that this system of classification may be adopted generally. Every major and minor series or type falls into its proper place in that system after a thorough field study and occasional laboratory tests have been completed.

Utility of Soil Surveys and Classification.—The obvious utility of soil analysis, research, surveys, and classification is to extend the horizon of knowledge and to make the best use of what has been gathered heretofore. Wherever a soil type is identified an intelligent judgment may be formed regarding its genesis and development, the climatic, rainfall, and drainage habits that prevailed, and the vegetation sustained during the progressive stages of its growth. In the brief period since soil science has been recognized adequately, it has rendered valuable service to agriculture in all its phases, to commerce, to the manufacturing industry, to water supply, to flood protection, and to the most extensive and important highway development yet undertaken.

What promises to be one of the most notable steps forward was inaugurated by the U. S. Bureau of Public Roads as sub-grade investigations. The work has been carried on in co-operation with various State highway departments and colleges. The net result thus far has been to narrow the problem down to the identification and treatment or the elimination of harmful elements. Inasmuch as the ordinary failures of pavements that are traceable in any manner to unstable sub-grades represent but a small proportion of the total mileage, attention has been concentrated on these sections. Further applications for soil science are yet to be realized in the broad fields of structural and foundation design, drainage, control of surface run-off and underground water supplies, crop production without serious depletion of fertility, and all lines of endeavor that depend on soil materials or properties.

Illustrative of the clues to soil characteristics which have long been familiar to even casual observers, a rank growth or a precarious one of either arrow-weeds or bud-sage, salt-grass or blue-grass, cactus or willows, may publish more information concerning the chemical analysis, texture, water supply, and drainage habits of the land than could be conveyed on several pages. If the principal vegetation is scrub cedar of the uplands, swamp cedar, or the giant redwood, radically different conditions are known to govern.

CLASSIFICATION OF PRECIPITATION PATTERNS

The time and labor of assembling and reducing the summarized precipitation characteristics presented in Table 1 (Appendix I) may prove to be meager as compared with what will be required to develop an adequate appraisal and interpretation of the data contained therein.

A tentative classification is presented in Table 5, in which the first or primary basis of division depends on climatological zones, approximately defined by parallels of latitude. The irregularities introduced by such influences as ocean currents and prevailing winds, as illustrated in Siberia, Labrador, and the Scandinavian Peninsula, for example, seem to be fully as significant and important as the similarities displayed by other groups of stations. The second and third categories have to do with distances inland and altitude, respectively; and, finally, the representative stations to show both the range of habits and also the average behavior for specified conditions, are arranged according to the amount of either annual or monthly rainfall. Other selections might be made to display somewhat greater or less concordance than appears in Table 5, wherein both length and completeness of record were taken into account.

Outstanding features of Table 1 (Appendix I), and Table 5 are the manifestations of progressive increase of moisture capacity with temperature. It is notable that the greatest observed intensity of rainfall occurs in the sub-tropical, and not in the equatorial regions. Evidently, this is not due to lack of atmospheric moisture, but to the fact that some of the conditions necessary to the violent relinquishing of moisture are not to be encountered near the equator. For example, the polar fronts and cold air currents and icebergs are all unheard of in lower latitudes for the same identical reasons. Special situations might be encountered, however, in which the condensing medium would be supplied from the sub-zero temperature zones in the upper atmosphere, if it were not for the regular daily relief afforded by the mid-day showers of the tropics; and it is conceivable that rates of precipitation might occur during a brief period as intense as have been registered elsewhere.

The tabulations plainly indicate that there are fairly definite limiting conditions and rates of rainfall appurtenant to the various climatological zones. There are typical rainfall patterns displayed in the percentage columns of Table 1 (Appendix I), which may be utilized for combining fragmentary records, or for supplying missing data under proper controls.

APPLICATION TO PRACTICAL PROBLEMS

In the solution of practical problems, the greatest needs are for more basic data and for a better means of interpreting those already available.

Inquiries as to what may be expected in the way of rainfall and run-off habits in a given locality are too often answered by reference to voluminous

TABLE 5.—REPRESENTATIVE PRECIPITATION STATIONS AND CHARACTERISTICS, FROM TABLE 1 (APPENDIX I).

Station No.	Station	Latitude zone.	Relation to coast.	Relative elevation.	Mean month.	Record of Observed Rainfall, in Inches.					Mean month.	Rainfall, in Percentage of Mean Annual Quantity.				
						Maximum.		Annual.				Maximum.		Annual.		
						24 hours.	Month.	Minimum.	Mean.	Maximum.		24 hours.	Month.	Minimum.	Mean.	Maximum.
784	Verkhoyansk, Siberia	Above 60°	Interior	Low	0.9	0.8	3.4	2.0	5.0	9.1	18	16	68	40	100	182
788	Obdorsk, Siberia	"	"	"	2.0	....	5.8	4.5	10.1	14.4	20	....	57	44	100	142
791	Bogoslovsk, Siberia	"	"	"	2.9	2.6	7.7	5.4	16.4	28.4	17	16	47	33	100	173
418	Dawson, Canada	"	"	High	1.2	....	4.2	9.2	14.4	17.7	8	....	29	64	100	123
782	Novo-Mariinski, Siberia	"	Coastal	Low	1.5	....	4.3	4.7	7.4	10.3	21	....	58	64	100	139
804	Gjesvar, Norway	"	"	"	3.3	....	10.4	17.3	29.1	46.1	11	....	36	59	100	158
811	Vestmanno, Iceland	"	"	"	5.8	....	13.9	37.3	53.7	70.7	11	....	26	69	100	131
313	Yuma, Ariz	30° to 60°	Interior	Low	0.6	4.0	6.2	0.6	3.4	11.4	17	117	182	18	100	335
185	St. Paul, Minn	"	"	"	4.1	5.7	11.7	10.2	27.2	49.7	15	21	43	37	100	183
142	Chattanooga, Tenn	"	"	"	4.8	7.6	15.3	32.7	51.3	70.6	9	15	30	64	100	138
266	Reno, Nev	"	"	High	1.6	2.7	6.8	4.7	8.2	13.7	20	33	83	57	100	167
256	Tamarack, Calif	"	"	"	11.6	....	39.8	27.8	50.6	94.3	23	....	79	55	100	187
638	Santis, Switzerland	"	"	"	8.7	....	27.5	53.3	100.3	154.2	9	....	27	53	100	154
236	San Diego, Calif	"	Coastal	Low	1.9	3.2	9.3	4.3	9.8	27.6	19	33	95	44	100	283
223	Fort Ross, Calif	"	"	"	11.4	....	37.4	21.7	51.9	88.2	22	....	72	42	100	170
361	Valdivia, Chile	"	"	"	16.3	....	34.0	71.7	106.3	140.7	15	....	32	67	100	132
570	O'okiep, South Africa	"	"	High	1.1	....	4.3	2.1	6.7	11.7	17	....	64	31	100	174
13	Mt. Washington, N. H	"	"	"	10.3	....	23.9	55.8	82.2	121.1	13	....	29	68	100	147
717	Ben Nevis, Scotland	"	"	"	19.1	7.3	48.3	107.8	160.8	240.1	12	4	30	67	100	150
611	Assuan, Egypt	10° to 30°	Interior	Low	0	0	0	0	0	0	....	....	....	....	....	....
601	Timbuktu, West Africa	"	"	"	2.8	....	....	....	7.7	....	36	....	....	....	100	....
378	Asuncion, Paraguay	"	"	"	6.2	....	18.6	22.6	51.8	78.5	12	....	36	44	100	151
549	Khartoum, Egypt	"	"	High	2.6	3.5	7.4	1.3	5.7	14.4	46	61	130	23	100	253
550	Deim Zubeir, Egypt	"	"	"	7.8	5.4	23.9	....	45.7	78.5	17	12	52	....	100	172
560	Belvedere South Africa	"	"	"	16.5	....	....	....	81.4	....	20	....	....	....	100	....
586	Swakopmund, South West Africa	"	Coastal	Low	0.1	....	0.8	0.1	0.6	1.2	17	....	133	17	100	200
108	Miami, Fla	"	"	"	9.1	9.0	27.9	33.1	60.0	89.1	15	15	46	55	100	148
511	Akyab, Burma	"	"	"	53.7	....	97.8	142.0	203.8	323.5	26	....	48	70	100	158
352	Arequipa, Peru	"	"	High	1.7	....	8.1	0.2	4.3	12.0	40	....	188	5	100	279
492	Baguio, Philippine Islands	"	"	"	40.3	46.0	133.0	125.7	182.0	355.8	22	25	73	69	100	196
515	Cherra Punji, India	"	"	"	107.4	40.0	366.1	282.6	456.9	905.+	23	9	80	62	100	198+
593	Gazi, Congo	0° to 10°	Interior	Low	8.5	....	....	....	72.7	....	12	....	....	....	100	....
594	Eala, Congo	"	"	"	8.8	....	....	....	70.5	....	12	....	....	....	100	....
552	Adis Ababa, Abyssinia	"	"	High	7.5	3.0	22.5	....	49.6	74.9	15	6	45	....	100	151
553	Entebbe, Uganda	"	"	"	8.5	....	15.8	47.2	58.0	75.7	17	....	27	81	100	131
551	Gore, Abyssinia	"	"	"	11.3	3.5	17.0	....	75.7	92.6	15	5	22	....	100	122
442	Malden Island, Pacific	"	Coastal	Low	4.7	....	25.7	3.9	28.6	93.6	17	....	90	14	100	327
554	Dar-es-Salaam, East Africa	"	"	"	11.8	....	23.8	19.4	42.3	56.9	28	....	56	46	100	134
602	Freetown, Sierra Leone	"	"	"	36.6	....	61.1	102.5	157.2	206.6	23	....	39	65	100	132
523	Nuwara Eliya, Ceylon	"	"	High	11.9	....	35.0	68.4	92.8	131.5	13	....	38	74	100	142
477	Kajoemas, East Indies	"	"	"	17.7	....	40.3	70.4	99.4	154.0	18	....	41	71	100	155

tabulations, when summarized data such as those in Table 1 (Appendix I) would serve more satisfactorily.

Arrangement of a long series of natural phenomena according to magnitude of the observed quantities, such as are used in precipitation and run-off records, invariably results in a crowding near the mean values, and a thinning out toward the extremes, suggesting a mass distribution like that of a cone or of a pyramid, in which the base represents the mean and the apex, either the maximum or the minimum. It is naturally expected that the longer the period of record, the wider is the interval between the mean and either the maximum or the minimum. Because the 500-year or other long-period maximum must fall in the record of some decade, care should be taken to investigate such a possibility. When such happenings are properly taken into account, some interesting and useful relations become apparent regarding periodic maxima and frequency curves:

First.—The only direct relations to be observed consistently among station records are such as may be ascribed either to contiguity or to similarity of climatic conditions.

Second.—No direct relation is traceable between rainfall and sunspot periodicity (11 to 12 years), which closely corresponds to the time of revolution about the sun (11.86 years) of the planet, Jupiter, representing 71% of the total known mass of the solar planetary system. Seven of the longest known annual precipitation records are platted in Fig. 2, for convenient comparison. The occasional similarities, contrasts, and synchronisms therein displayed, seem to indicate endless variety rather than conformity to any generalization.

Third.—The mean monthly or the mean annual precipitation, together with the maxima for corresponding periods to be expected during average intervals of 10 years, 50 years, and 250 years, seems to form a simple arithmetical progression, while the related time intervals, namely, 2, 10, 50, and 250 years, form a geometrical progression. Fig. 3 illustrates for the yearly precipitation how nearly the straight-line relation holds for frequency curves when they are thus plotted on semi-logarithmic paper. Similar results are found for monthly periods.

Fourth.—The ten percentage columns of Table 1 (Appendix I) seem to provide generally ascending series of either geometrical or arithmetical progression, from left to right. The quantities in Column (1), headed "Mean Daily Intensity on Rainfall Days", are but the reciprocals of the average number of rainy days per year, expressed in percentage. Thus, for stations with 90 days of rainfall per annum, the mean daily intensity is 1.1% of the total mean annual precipitation. The quantities in Column (1) seem to be significant and important when considered in connection with the remaining nine columns. In general, this percentage is numerically equal to the maximum rainfall to be expected in a 3-min. period.

Since the beginning of these investigations and the assembling of the foregoing data, many opportunities have been presented for practical tests of methods herein advocated. For example, preliminary estimates were made for the waterways required in connection with current projects, mainly from consideration of rainfall habits and soil texture, classification, and covering. Later, they were compared with the final estimates, made after detailed study of the water-shed, run-off records, and all other available data, with satisfactory results.

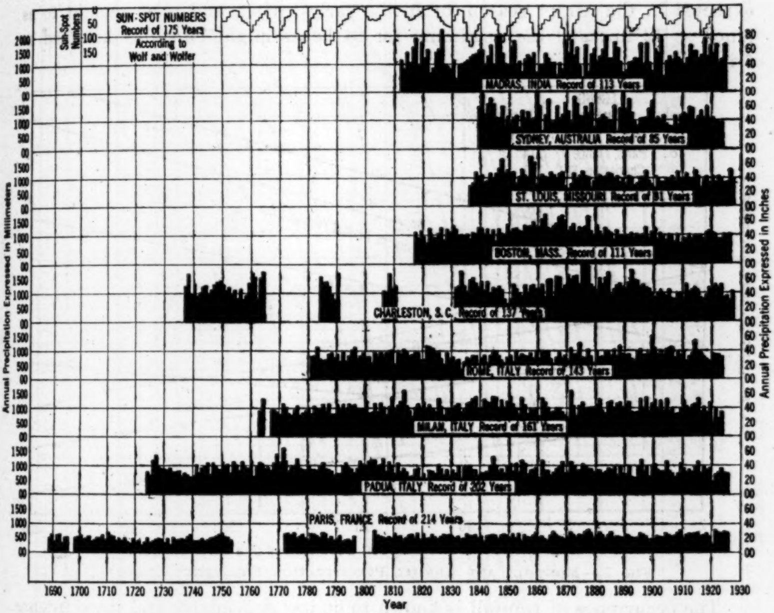


FIG. 2.—PRECIPITATION RECORDS AT NINE TYPICAL STATIONS, SHOWING RELATION TO SUNSPOT NUMBERS

Application to a foreign problem is afforded by the Habra Dam disaster of November 26, 1927, in the Province of Oran, Northern Algeria. This dam had previously failed on March 10, 1872, in the year following its construction; and again on December 16, 1881, after reconstruction and enlargement of the spillway, the fatalities in this instance amounting to 400.[5]

The scarcity or utter absence of meteorological and hydrographic records are admittedly among the primary causes of the recurrent disasters. Through costly experience it has now been demonstrated that the 3 085 sq. miles of water-shed above the dam site are reasonably capable of yielding run-off at the rate of 20 sec-ft. per sq. mile instead of only one-fourth or one-half of that amount as assumed for the original construction and reconstruction, respectively.

---

[5] "The Failure of the Habra Dam, Algeria," *Le Génie Civil*, March 17, 1928, p. 256.

There is enough similarity observable in the rainfall patterns for Italian, French, and Spanish stations near the Mediterranean Coast, to permit expansion of the records for North Africa by use of such definite data as have long been available for the Ports of Tunis, Algiers, and Tangiers. The annual rainfall to be expected in the vicinity of Oran and Perregaux, in the Habra Basin, is known to range usually from 20 to 28 in.; the monthly maximum is to be expected in the winter season, or in the early spring, and may amount to as much as 12 in.; and the maximum for 24 hours might be safely assumed as 6 in.

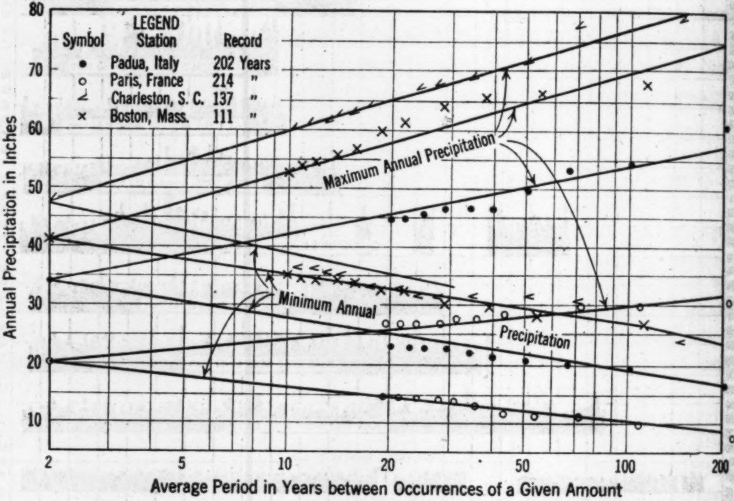


FIG. 3.—MONTHLY AND ANNUAL PRECIPITATION FREQUENCY CURVES

The occurrence of rainfall is known to be less dependable and more intense in Algeria than on the northern shores of the Mediterranean, just as should be expected when the desert character of the hinterland is taken into account. Finally, the Tiber River in Italy and the rivers of Eastern Spain should have afforded an index as to the expected run-off from the Habra Basin capable of governing an adequate design for the spillway that has recently failed for the third successive time.

Illustrative of the recommended method of approach for such practical problems, the meteorological records of the most nearly adjacent and comparable stations were reviewed in their summarized form as used in Table 1 (Appendix I), and supplemented by such literature and data as were available for the locality, in order to form a fair conception of the run-off habits to be expected under extreme conditions, as well as the average prevailing in that locality. Other published data[6] afford a number of examples which may be

---

[6] "Flood Flow Characteristics," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1003, Table 2 (Appendix I).

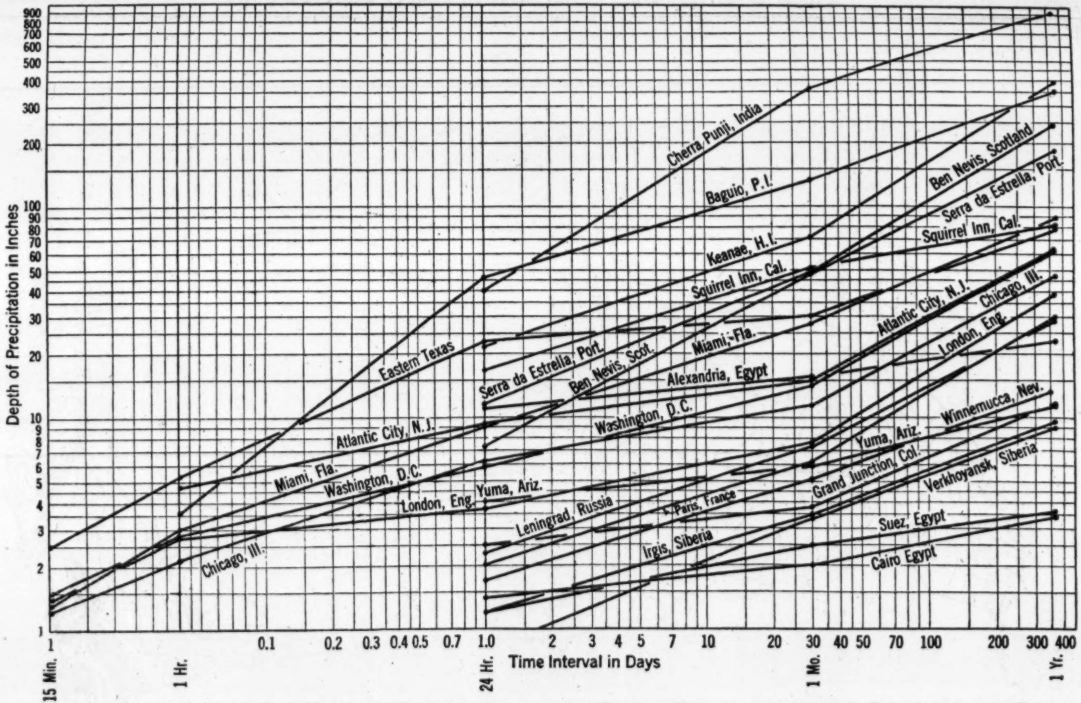


FIG. 4.—CHART OF MAXIMUM RAINFALL OBSERVED AT TWENTY-TWO TYPICAL STATIONS FOR VARIOUS TIME INTERVALS (BASED ON DATA IN TABLE 1 (APPENDIX I)).

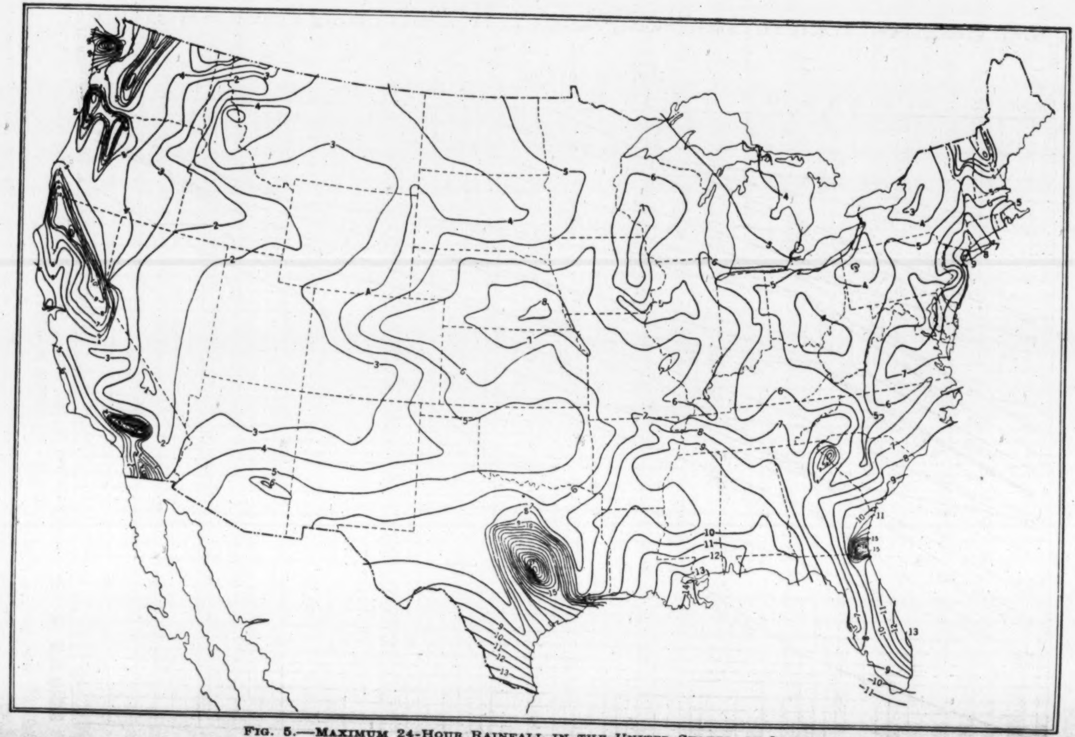


FIG. 5.—MAXIMUM 24-HOUR RAINFALL IN THE UNITED STATES, IN INCHES.

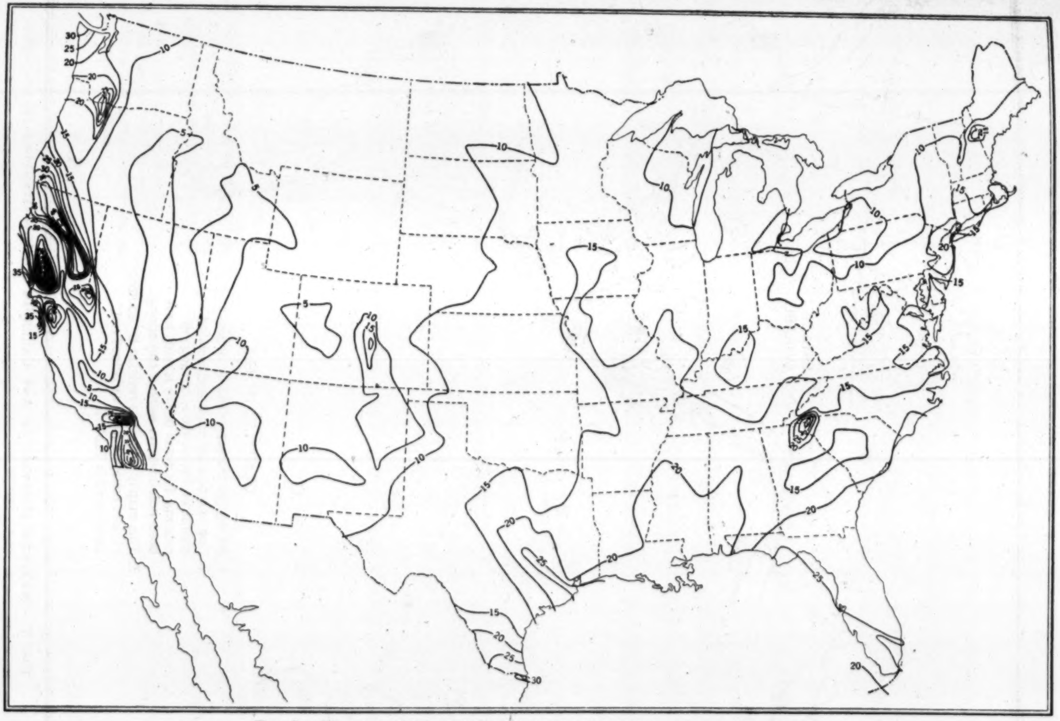
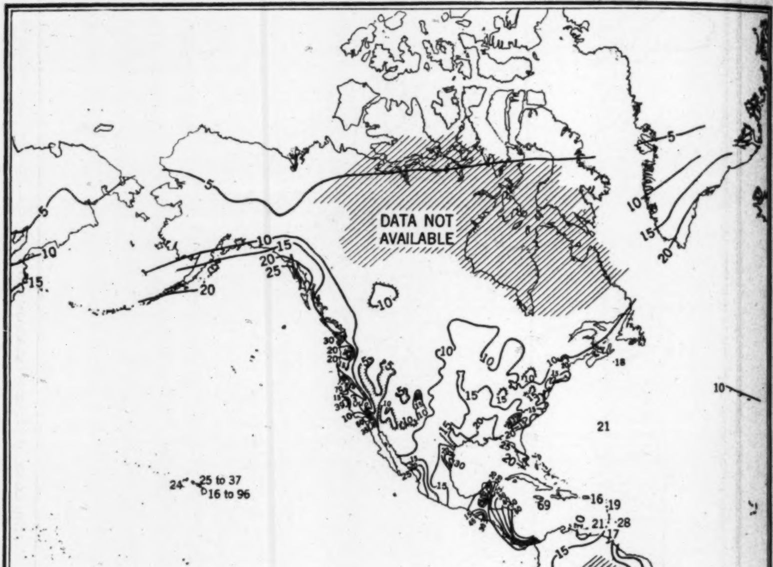

5
10
15
20
25
30
35

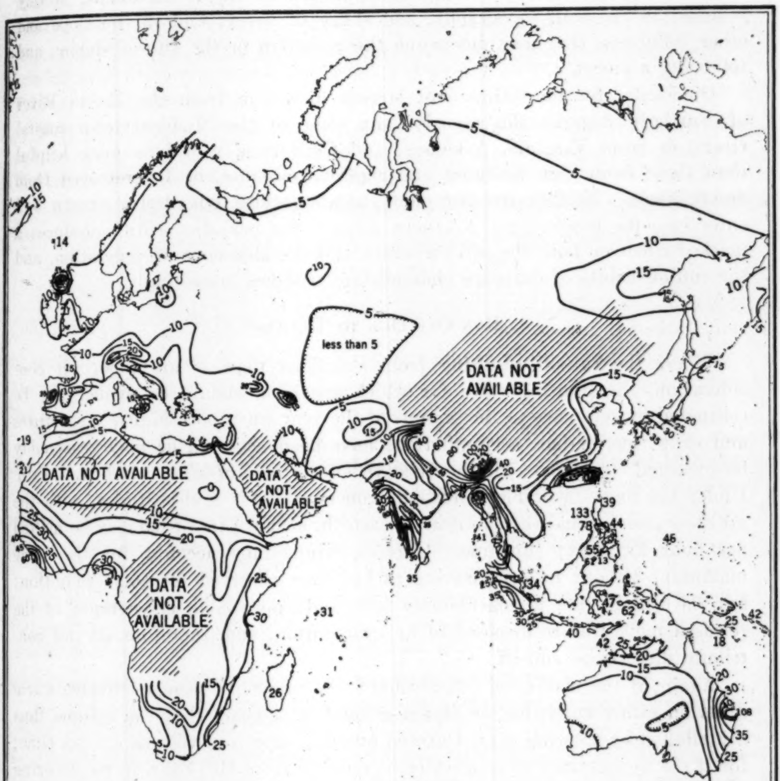
FIG. 6.—MAXIMUM MONTHLY RAINFALL IN UNITED STATES, IN INCHES.


DATA NOT AVAILABLE
DATA NOT AVAILABLE
5
10
15
20
25
30
24
25 to 37
16 to 96
21
18
16
19
21
28
17
69
30
42
40
37
19
38
42
6
14
3
10

GENERAL NOTES

1. All depths are expressed in inches.
2. Due to restricted scale and isolated character, the maxima, or range of maxima, for small ocean islands have been noted hereon without contouring.
3. This map is subject to revision by addition of data covering local variations not considered in Table I.

FIG. 7.—MAXIMUM RAINFALL IN ANY CALENDAR MONTH—WESTERN HEMISPHERE.


DATA NOT AVAILABLE
DATA NOT AVAILABLE
DATA NOT AVAILABLE
DATA NOT AVAILABLE
less than 5
5
10
15
20
25
30
35
14
19
31
26
133
78
39
44
55
46
47
62
42
40
18
108
34

GENERAL NOTES

1. All depths are expressed in inches.
2. Due to restricted scale and isolated character, the maxima, or range of maxima, for small ocean islands have been noted hereon without contouring.
3. This map is subject to revision by addition of data covering local variations not considered in Table I.

FIG. 7.—MAXIMUM RAINFALL IN ANY CALENDAR MONTH—EASTERN HEMISPHERE.

utilized for comparisons if due allowance be made for differences in the temperature, humidity, seasonal habits, and wind directions during stormy periods; in the soil, topography, and shape of basin; and in the slope and other influences that may determine the retention or the run-off during and following a storm.

Obviously, for an estimate of stream discharge from the Habra River of Northern Algeria, the records from some of the Mediterranean coastal rivers, or from Virginia, Arkansas, and California, would be more helpful than those from high latitudes and rigorous climates, or far removed from desert areas. Neither ice nor snow accumulations nor deeply frozen soil enter into the problem for Northern Africa; the periodic aridity produces a marked effect on both the soil structure and the character of vegetation, and the run-off habits of drainage channels are modified accordingly.

Run-Off Due to Thawing

Where the run-off resulting from a sudden thaw is an important consideration, the limiting rates for which provision should be made, may be estimated by reference to the seasons of the year when both high temperatures and snow, ice, or hail accumulations have been observed or may reasonably be expected, together with the relative effects of sunshine, wind, and rain. Under the most favorable conditions conceivable for mid-latitudes, with the air clear and dry and the sun near the zenith, nearly ½ in. of ice may be melted per hour by direct sunshine. Chinook winds may account for the same maximum 24-hour rate of melting as has been observed for solar radiation; but the hourly rate is considerably less, and appreciable percentages of the released moisture are disposed of by evaporation, and, therefore, do not contribute directly to run-off.

Again, if the maximum conceivable flood conditions seem to involve warm rains on either snow, ice, or frozen ground, it appears that the volume thus contributed by thawing cannot exceed one-third the rain volume at that time; for if the temperature of rain water be reduced from 80° Fahr. to the freezing point by contact with ice, it would release not more than one-third its weight of frozen moisture, as determined by the ratio of the 48 heat units extracted to the 144 required for melting ice, to which reference has already been made; and the assumed limits are seldom if ever attained.

Each application of the principles and methods herein set forth should receive individual treatment; but the controls are essentially collective and require mature judgment, observation, and experience for the best results.

Graphical Representation of Rainfall Characteristics

The maximum rates of rainfall that have been observed as typical stations representing almost the entire range of habits for the stations listed in Table 1 (Appendix I), are illustrated in Fig. 4. The platting on multi-phase logarithmic diagrams brings out not only the similarities, but also the divergencies of precipitation patterns and trends. Figs. 5 and 6 portray the maximum observed depths of precipitation in 24 hours and in one calendar month, respectively, for the United States, based on the data of Table 1 (Appendix I). Fig. 7 is a graphical representation of the maximum monthly rainfall over the entire earth so far as is disclosed by the data contained in Table 1 (Appendix I).

It is readily recognized that the inclusion of additional station records and the use of a larger scale for mapping might introduce important revisions. The curves drawn on the Mercator projection, however, may serve as generalizations and may assist in visualizing some of the rainfall characteristics of the world.

CONCLUSIONS

1.—The precipitation to be reasonably expected at a given place, together with the run-off resulting therefrom, may be regarded as but the expression of certain physical forces and habits, explainable in the main on the basis of known scientific laws and principles applied to the source of supply and to the paths traversed by winds and drainage channels.

2.—The factors responsible for both rainfall and run-off are fairly well defined; most of the earth's surface has been explored; the courses of ocean currents and the prevalent paths of storms have been charted; and the potentialities of recurrent storm movements have been recorded in terms of barometric pressures, precipitation, wind velocities, and flood volumes.

3.—Wherever meteorological and hydrographic records are available, approximations may be made with varying degrees of dependability as to both the ordinary habits and the extremes of behavior against which provision should be made. By utilizing the long-period records as controls, the brief or fragmentary data may be interpreted more effectively.

4.—Precipitation and run-off habits which govern the design of waterways display characteristics definite enough to be classified, and they are known or may be derived from well co-ordinated records within nearly the same range of accuracy as is ordinarily attained for strength of structural materials.

5.—Many of the catastrophes resulting from storms and floods should be recognized as preventable accidents, and public works should be inaugurated to reduce such hazards.

APPEN-

TABLE 1.—OBSERVED RA
(Depths Expre

Station No.	Station.	Elevation above sea, in feet.	Years of Record.			Maximum Observed Precipitation.					Annual Precipitation.			Observed Precipitation, in Percentages of Mean Annual Rainfall.										
			For day or less.	For month and year.	Last year included.	Number of Minutes.			Day of 24 hours.	Month.	Minimum.	Mean.	Maximum.	Mean daily intensity on days of rainfall.	Maximum During Short Periods. Number of Minutes.			Mean for month of maximum record.	Maximum.		Annual.			
						5.	15.	60.							5.	15.	60.		In 24 hours.	In one calendar month.	Minimum.	Mean.	Maximum.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
1	Eastport, Me.	53		56	1927					13.2	22.8	40.4	64.5	0.7				7.8		32.6	57	100	160	2; 3
2	Bar Harbor, Me.	20		44	1920					14.4	32.1	48.3	63.9	0.8				10.3		29.8	66	100	132	5
3	Orono, Me.	129		56	1925					10.5	26.0	41.7	53.8	0.9				7.7		25.2	62	100	129	5
4	Upper Oxford Dam, Me.	1 470		40	1925					8.8	22.4	32.8	41.7					10.1		26.8	68	100	127	5
5	Lewiston, Me.	185		52	1927					12.3	32.7	44.2	58.6					9.8		27.8	74	100	133	2; 3
6	Gardiner, Me.	163		90	1927					13.1	30.2	43.0	54.6					9.5		30.5	70	100	127	2; 3
7	Portland, Me.	99	57	57	1927	0.42	1.09	1.58	5.3	10.9	32.0	41.7	59.7	0.8	1.0	2.6	3.8	7.9	12.7	26.2	77	100	143	1
8	Cornish, Me.	778		67	1924					12.3	33.4	46.1	63.3					8.1		26.6	72	100	138	5
9	Lakeport, N. H.	500		68	1925					12.6	31.5	42.2	55.6					9.1		29.9	75	100	132	5
10	Concord, N. H.	350		70	1927	0.55	1.55	2.48	4.1	11.6	26.2	38.4	54.8	0.8	1.4	4.0	6.5	8.7	10.7	30.2	68	100	141	5; 3
11	Hanover, N. H.	603		61	1927					9.8	22.7	35.9	55.8	0.8				9.6		27.5	63	100	155	5; 3
12	Plymouth, N. H.	500		40	1927					11.5	30.8	39.3	53.5					8.9		29.3	78	100	136	2; 3
13	Mt. Washington, N. H.	6 293		21	1890					23.9	56.8	82.2	121.1					12.6		29.0	68	100	147	2
14	Bethlehem, N. H.	1 470		33	1925					9.8	28.0	36.7	46.1	0.7				10.4		26.6	76	100	125	5
15	Burlington, Vt.	404	22	93	1927	0.44	0.78	1.52	4.5	9.9	24.3	32.6	49.4	0.7	1.3	2.4	4.7	10.6	13.7	30.4	74	100	152	5; 3
16	Northfield, Vt.	876		40	1927	0.85	1.64	1.91	7.6	11.4	27.4	33.2	45.9	0.6	2.6	5.0	5.8	5.5	22.9	34.4	82	100	138	2; 3
17	Chelsea, Vt.	840	30	42	1927				4.5	10.9	27.1	35.8	48.1					8.7	12.5	30.5	76	100	134	2; 3
18	Woodstock, Vt.	700		60	1927					12.9	29.7	38.2	51.3					8.0		33.8	78	100	134	2; 3
19	Williamstown, Mass.	711		50	1926					14.8	30.5	38.1	54.5	0.9				11.2		28.8	80	100	143	5; 3
20	Springfield, Mass.	200		79	1926					15.0	30.9	43.7	58.0					9.9		34.4	71	100	133	5; 3
21	Amherst, Mass.	222		91	1926					14.5	31.0	43.9	58.0	0.8				9.9		33.0	71	100	132	5; 3
22	Lowell, Mass.	80		101	1926					12.3	27.8	41.3	59.5					9.9		29.8	67	100	144	5; 3
23	Boston, Mass.	124	57	110	1927	0.55	1.12	1.64	6.0	12.4	27.2	43.5	67.7	0.8	1.3	2.6	3.8	8.2	13.8	28.5	63	100	158	2
24	Cambridge, Mass.	40		165	1927						24.5	42.3	62.5								58	100	148	5
25	Cambridge, Mass (Harvard Obs)	75		81	1921					15.0	30.1	43.5	59.3					9.5		34.5	69	100	137	5
26	Cambridge, Mass (Fresh Pond)	40		40	1925					11.7	28.2	41.0	62.5					8.5		28.6	69	100	153	5
27	Waltham, Mass.	30		101	1925					14.0	28.4	42.3	62.1					9.7		33.2	67	100	147	5
28	Lake Cochituate, Mass.	140		74	1925					13.3	31.2	45.6	69.3					8.5		29.1	68	100	152	5; 5
29	New Bedford, Mass.	100		113	1926					18.7	32.9	46.1	65.4					9.0		40.6	71	100	142	5; 3
30	Nantucket, Mass.	15	33	52	1926	0.44	0.83	2.26	5.7	11.0	27.1	39.3	58.0	0.7	1.1	2.1	5.8	8.2	14.5	28.0	69	100	148	5; 3; 11
31	Block Island, R. I.	26		46	1926	0.21	0.52	1.53		9.8	27.2	41.2	63.1	0.8	0.5	1.2	3.7	8.4		23.8	66	100	153	5; 3; 11
32	Providence, R. I.	162	22	95	1920	0.45	1.11	1.98	4.8	11.3	30.5	45.1	63.5	0.8	1.0	2.5	4.4	8.5	10.8	25.0	68	100	141	5; 1; 3
33	Norwich, Conn.	150		55	1925					13.3	29.8	44.4	62.8					9.8		29.8	67	100	136	5
34	New London, Conn.	47		56	1926					16.4	30.0	44.1	60.6					9.6		37.2	68	100	137	5; 2; 3
35	Middletown, Conn.	125		67	1925					15.5	34.0	46.8	66.7					8.5		33.1	73	100	142	5
36	Hartford, Conn.	159		55	1926	0.49	1.30	2.64		15.3	33.5	44.7	61.0	0.8	1.1	2.9	5.9	9.7		34.2	75	100	136	1
37	Canton, Conn.	900		64	1922					18.0	34.8	46.8	75.2					8.6		38.8	71	100	154	5
38	New Haven, Conn.	127		76	1926	0.40	1.05	2.33		17.1	33.9	45.6	60.3	0.8	0.9	2.3	5.1	9.4		37.5	74	100	132	5; 2; 3
39	New York, N. Y.	314	26	56	1927	0.75	1.63	2.48		14.5	33.2	43.1	58.7	0.8	1.7	3.8	5.8	7.9		33.7	77	100	136	1
39-A	New York, N. Y.	314	56	101	1927				9.4	14.5	28.8	42.1	58.7	0.8				8.0	22.3	34.4	68	100	139	2; 1
40	Yonkers, N. Y.	100		48	1925					22.7	25.2	48.4	85.2					10.4		47.0	52	100	176	5
41	West Point, N. Y.	167		76	1925					13.5	31.5	46.3	63.6	1.0				8.1		29.8	70	100	141	2; 5
42	Boyd's Corners, N. Y.	600		58	1925					14.4	35.3	49.0	63.4					8.6		29.4	72	100	129	2; 5
43	Albany, N. Y.	97	45	101	1926	0.57	1.17	1.83	4.7	13.5	26.4	37.4	56.8	0.7	1.5	3.1	4.9	8.7	12.5	36.0	71	100	152	2; 5; 1
44	Plattsburgh, N. Y.	183		73	1920					9.2	18.8	30.4	47.3	1.0				11.6		30.3	62	100	155	2
45	Canton, N. Y.	448	15	38	1920				3.5	8.9	28.3	36.0	46.2					9.5	9.7	24.7	78	100	128	2
46	Oswego, N. Y.	335	50	74	1920				3.8	9.8	23.4	36.5	56.1	0.6				8.9	10.4	26.8	64	100	154	2
47	Syracuse, N. Y.	597	19	24	1920	0.56	1.16	2.66	4.8	7.4	29.2	34.7	44.5	0.8	1.6	3.3	7.6	9.5	13.8	21.3	84	100	128	2; 15
48	Rochester, N. Y.	500	50	92	1920				4.2	8.7	17.0	33.2	49.9	0.6				8.8	12.7	28.3	51	100	151	2
49	Fort Niagara, N. Y.	300		48	1899					7.3	16.4	27.5	50.5					10.2		28.6	60	100	184	2
50	Buffalo, N. Y.	824		71	1927	0.79			4.2	10.6	25.3	36.1	60.2	0.6	2.2			8.3	11.6	29.3	70	100	167	2; 1; 9
51	Jamestown, N. Y.	1 368		28	1918					11.2	34.4	43.3	52.7	0.8				11.0		25.9	79	100	122	2
52	Bolivar, N. Y.	1 600		30	1920					11.0	31.8	40.1	48.6					9.7		27.4	79	100	122	2
53	Ithaca, N. Y.	928	42	64	1915				4.2	8.4	20.2	32.9	46.4	0.6				9.7	12.8	25.6	62	100	141	2
54	Cooperstown, N. Y.	1 200		67	1920					10.0	29.9	40.9	58.1	0.7				10.6		24.4	73	100	142	2
55	Binghamton, N. Y.	871		38	1927		1.00	2.03	4.5	8.1	22.4	34.0	42.6	0.7		2.9	5.9	10.3	13.3	23.7	66	100	125	1
56	Scranton, Pa.	805		33	1927				5.1	9.2	31.3	37.3	45.3	0.7				8.3	13.7	24.8	84	100	122	1; 2
57	Philadelphia, Pa.	156	46	56	1927			2.35	5.9	12.1	29.3	41.2	55.3	0.7			5.7	11.2	14.5	29.4	71	100	134	1; 2
58	Philadelphia, Pa.	156	46	101	1920			2.35	5.9	15.8	29.3	42.7	61.2	0.7			5.5	10.5	13.8	37.0	69	100	143	2
59	West Chester, Pa.	455	59	72	1920				6.6	14.6	33.0	49.1	73.0	0.7				9.8	13.4	29.7	67	100	149	2
60	Reading, Pa.	325	30	60	1927				5.4	13.3	31.5	40.8	65.0	0.8				8.0	13.2	32.6	77	100	159	2; 1
61	Lebanon, Pa.	469	26	82	1920				5.2	12.9	28.9	44.4	64.1	0.8				10.4	11.7	29.0	65	100	145	2
62	Harrisburg, Pa.	361	32	55	1920				6.2	13.6	26.0	39.4	57.1	0.8				9.7	15.8	34.7	66	100	145	2
63	Altoona, Pa.	1 615	25	37	1920				4.3	9.8	21.2	34.3	44.4					11.4	12.5	28.6	62	100	129	2
64	Johnstown, Pa.	1 184	28	40	1920				4.5	12.2	37.5	47.8	58.7	0.6				9.6	9.4	25.5	78	100	123	2
65	Freeport, Pa.	750		44	1920					8.9	30.8	42.0	57.9					7.9		21.3	73	100	138	2
66	Franklin, Pa.	955	25	55	1920				2.8	11.6	34.0	42.5	59.7	0.6				10.5	6.6	27.3	80	100	140	2
67	Erie, Pa.	714	46	48	1920				5.8	9.3	26.7	37.9	55.0	0.6				8.9	15.4	24.6	70	100	145	2
68	Pittsburgh, Pa.	842	57	57	1927	0.71	1.20	1.65	4.1	9.5	25.7	36.2	50.6	0.6	2.0	3.3	4.6	11.6	11.3	26.3	71	100	160	1
69	Pittsburgh, Pa.	842	57	91	1927	0.71	1.20	1.65	4.1	9.5	25.3	36.2	50.6	0.6	2.0	3.3	4.6	10.7	11.3	26.3	70	100	160	2; 3
70	Somerset, Pa.	2 280		42	1920				4.4	14.3	32.3	51.2	66.3	0.7				9.8	8.6	28.0	63	100	130	2
71	Morgantown, W. Va.	1 250		51	1923					11.2	28.1	43.3	69.5	0.9				11.5		28.8	65	100	161	2; 3
72	Parkersburg, W. Va.	638	25	43	1927	0.75	1.48	2.12	3.8	11.5	24.3	39.5	62.7	0.7	1.9	3.7	5.4	11.2	9.6	29.0	62	100	158	1; 3
73	Weston, W. Va.	1 026		37	1924					15.1	31.2	47.3	68.2	0.8				10.6		31.9	66	100	144	2; 3
74	Elkins, W. Va.	1 962	23	29	1927	0.66	1.05	2.37	4.3	11.1	38.8	47.1	65.4	0.6	1.2	2.2	5.1	11.7	9.2	23.5	82	100	139	1; 3
75	Washington, D. C.	112	57	85	1927	0.77	1.51	2.79	5.8	14.4	16.8	40.5	61.3	0.8	1.9	3.7	6.9	10.0	14.4	35.5	46	100	152	4; 3; 1
76	Washington, D. C.	112	53	53	1923	0.77	1.51	2.79	5.8	14.4	30.8	42.2	61.3	0.8	1.8	3.6	6.6	9.8	13.8	34.2	73	100	145	1
77	Baltimore, Md.	115	57	111	1927				6.1	12.3	22.4	40.3	62.3	0.8				9.9	15.1	30.6	56	100	155	2; 1
78	Baltimore, Md.	115	57	57	1927				6.1	12.3	31.6	42.3	62.3	0.8				10.4	14.4	29.1	75	100	147	1
79	Trenton, N. J.	60	28	43	1913			1.70	5.4	15.2	34.3	48.8	67.2	1.0			3.5	10.6	11.1	31.1	70	100	138	2
80	Newark, N. J.	159		78	1920					22.5	34.5	47.8	69.1	0.8				10.6		47.0	72	100	145	2
81	Atlantic City, N. J.	16	48	48	1920			4.73	9.2	14.9	28.1	40.5	61.1	0.8			11.6	10.7	22.7	36.8	69	100	151	2
82	Fortress Monroe, Va.	8		55	1890					18.0	19.3	42.6	74.1	1.0				10.9		42.2	45	100	174	2
83	Norfolk, Va.	91	57	57	1927	0.66	1.37	2.72	6.3	11.9	30.4	45.6	70.7	0.8	1.4	3.0	6.0	7.4	13.8	26.1	67	100	155	1
84	Richmond, Va.	144	22	57	1927	0.70	1.55	2.97	7.3	14.0	27.6	41.6	72.0	0.8	1.7	3.7	7.1	11.0	17.6	33.6	66	100	173	1
85	Charlottesville, Va.	854		45	1920					14.4	30.1	44.5	65.3	0.9				10.4		32.1	67	100	146	2
86	Lexington, Va.	1 060		58	1920					15.9	30.2	40.2	60.0	0.9				8.6		39.5	75	100	149	2
87	Lynchburg, Va.	681	17	50	1920	0.56	1.51	3.49	7.2	12.4	30.5	41.7	60.6	0.8	1.3	3.6	8.4	9.8	17.2	29.7	73	100	145	2
88	Danville, Va.	413		30	1920					11.9	32.0	42.1	54.9	1.0				11.1		28.2	76	100	130	2
89	Wytheville, Va.	2 293	18	48	1920				4.2	13.1	30.9	40.4	62.6	0.8				10.6	10.4	32.5	76	100	155	2
90	Asheville, N. C.	2 255	18	19	1920	0.83	1.71	2.70	7.92	11.7	30.1	39.9	51.1	0.8	2.1	4.3	6.8	11.8	19.9	29.8	75	100	128	2
91	Murphy, N. C.	1 614		49	1920					15.4	45.6	59.2	84.8	0.9				8.0		26.0	77	100	143	2
92	Rock House, N. C. Next to Maxima	3 100		29	1920					36.4	62.9	84.3	113.8 110.6	0.8				11.2		43.1	74	100	135	2
93	Charlotte, N. C.	779	18	44	1922	0.59	1.12	1.98	5.0	16.5	34.8	46.9	68.4	0.8	1.3	2.4	4.2	11.3	10.6	35.2	74	100	146	2
94	Raleigh, N. C.	390	41	41	1927	0.75	1.74	3.01	5.7	13.6	32.1	46.4	63.4	0.8	1.6	3.7	6.5	11.8	12.3	29.3	69	100	138	1
95	Hatteras, N. C.	11		51	1925					16.3	29.6	54.9	102.0	0.7				10.4		29.7	54	100	188	4
96	Wilmington, N. C.	52	50	50	1920	0.66	1.06	2.25	8.0	21.1	27.7	48.8	83.6	0.8	1.4	2.2	4.6	13.4	16.4	43.2	57	100	171	2
96-A	Effingham, S. C.	106		30	1920				13.2	24.9	34.3	48.9	70.1					11.8	27.0	50.9	70	100	144	2
97	Charleston, S. C.	48	23	136	1927	0.56	1.25	2.72	9.5	19.2	23.7	47.5	78.4	0.9	1.2	2.6	5.7	13.8	20.0	40.4	50	100	165	2; 1
98	Columbia, S. C.	351	20	48	1920	0.74	1.28	2.28	4.9	10.1	30.9	44.0	63.2	0.9	1.7	2.9	5.2	7.6	11.2	23.0	70	100	144	2
99	Camden, S. C.	222		72	1920					14.9	30.1	44.5	57.3	1.0				11.8		38.5	68	100	129	2
100	Greenville, S. C.	1 039	11	35	1920	0.52	1.23	3.43	8.2	19.5	42.7	52.0	77.8	0.9	1.0	2.4	6.6	9.7	15.8	37.5	82	100	149	2
100-A	Anderson, S. C.	764	26		1920				11.6	18.4	38.4	47.4	64.3					11.3	24.5	38.8	81	100	136	2
101	Augusta, Ga.	180	18	66	1920	1.24	2.11	3.08	5.6	14.0	35.8	46.5	60.7	0.9	2.7	4.5	6.6	10.6	12.1	30.1	77	100	130	2
102	Atlanta, Ga.	1 218	48	48	1926				7.4	15.8	33.1	48.8	65.2	0.9				10.3	15.2	32.4	68	100	134	1
103	Macon, Ga.	370	28	53	1926				6.8	13.4	31.7	44.9	61.9	0.9				10.8	15.2	29.8	71	100	138	2; 1
104	Savannah, Ga.	65	24	57	1927	0.72	1.21	3.56	10.1	22.9	32.6	48.5	73.9	0.9	1.5	2.5	7.4	11.2	20.8	47.2	67	100	152	1
104-A	St. George, Ga.	76		15	1920				18.0	24.2	30.9	49.6	63.2					17.7	36.3	46.8	62	100	128	2
105	Jacksonville, Fla.	222	33	56	1926	0.78	1.65	3.13	9.9	21.8	32.6	50.7	82.0	0.8	1.5	3.3	6.2	14.6	19.6	43.1	64	100	162	1
106	St. Augustine, Fla.	10		62	1923					23.9	31.6	47.7	71.8	1.1				13.6		50.0	66	100	150	2
107	Jupiter, Fla.	21	24	25	1920	0.65	2.12	3.83	13.2	21.4	39.8	59.5	87.1	0.7	1.1	3.6	6.4	17.8	22.2	36.0	67	100	146	2
108	Miami, Fla.	83	16	45	1926	0.58	1.34	3.05	9.0	27.9	33.1	60.0	89.1	0.8	1.0	2.2	5.1	15.2	15.0	48.5	55	100	148	1
109	Key West, Fla. Next to Maxima	16	50	77	1920	0.65	1.45	3.35	11.9	19.8	20.5	37.3	69.7	0.9	1.7	3.9	9.0	13.7	31.9	53.0	55	100	187	2
110	Tampa, Fla.	103	31	68	1927	1.00	1.90	3.30	6.6	24.5	32.2	50.2	89.9	0.8	2.0	3.8	6.6	17.8	13.1	45.8	64	100	179	2; 1
111	Apalachicola, Fla.	24	24	24	1926	0.78	1.65	2.72	10.1	27.7	30.7	57.4	77.8	0.8	1.4	2.9	4.7	14.8	17.6	48.2	53	100	135	1
112	Pensacola, Fla.	151	21	48	1927	0.78	2.27	4.27	10.7	18.6	35.6	57.4	86.0	0.9	1.4	3.9	7.4	13.6	18.6	32.4	62	100	150	1
113	Mobile, Ala.	84		67	1927					26.7	39.5	61.8	92.8	0.8				8.9		43.2	64	100	149	1
114	Montgomery, Ala.	240	56	56	1927				10.0	17.8	37.0	51.7	77.9	0.9				10.1	19.4	34.5	72	100	151	1
115	Birmingham, Ala.	701		40	1927				8.8	20.1	34.3	52.2	76.2	0.8				10.2	16.8	33.5	66	100	145	1
116	Florence, Ala.	574		37	1920					16.1	40.7	51.2	68.4	1.1				11.2		31.4	79	100	134	2
117	Meridian, Miss.	375	32	34	1920	0.62	1.53	3.66	9.5	20.1	34.9	52.7	73.4	0.9	1.2	2.9	7.0	8.8	18.0	38.2	68	100	140	1
118	Vicksburg, Miss.	247	41	60	1920	0.83	1.41	3.11	8.0	22.2	38.0	52.0	84.2	0.9	1.6	2.7	6.0	10.2	15.4	42.7	73	100	162	2

DIX I.

FALL FOR VARIOUS PERIODS.

ed in Inches.)

Station No.	January. Mean month. (26)	January. Maximum. 24 hours. (27)	January. Maximum. Month. (28)	February. Mean month. (29)	February. Maximum. 24 hours. (30)	February. Maximum. Month. (31)	March. Mean month. (32)	March. Maximum. 24 hours. (33)	March. Maximum. Month. (34)	April. Mean month. (35)	April. Maximum. 24 hours. (36)	April. Maximum. Month. (37)	May. Mean month. (38)	May. Maximum. 24 hours. (39)	May. Maximum. Month. (40)	June. Mean month. (41)	June. Maximum. 24 hours. (42)	June. Maximum. Month. (43)	July. Mean month. (44)	July. Maximum. 24 hours. (45)	July. Maximum. Month. (46)	August. Mean month. (47)	August. Maximum. 24 hours. (48)	August. Maximum. Month. (49)	September. Mean month. (50)	September. Maximum. 24 hours. (51)	September. Maximum. Month. (52)	October. Mean month. (53)	October. Maximum. 24 hours. (54)	October. Maximum. Month. (55)	November. Mean month. (56)	November. Maximum. 24 hours. (57)	November. Maximum. Month. (58)	December. Mean month. (59)	December. Maximum. 24 hours. (60)	December. Maximum. Month. (61)
1	3.78	...	9.0	3.24	...	9.4	3.88	...	9.4	2.90	...	6.8	3.16	...	12.2	3.11	...	7.4	3.18	...	9.1	3.23	...	9.4	2.91	...	6.4	3.91	...	9.5	3.58	...	8.2	3.58	...	8.8
2	4.93	...	11.1	4.50	...	9.1	4.96	...	14.4	3.60	...	11.0	3.33	...	10.8	3.30	...	10.3	3.41	...	7.9	3.40	...	8.2	3.70	...	9.8	4.29	...	9.5	4.81	...	9.2	4.44	...	9.8
3	3.86	...	8.1	3.49	...	8.4	3.79	...	8.9	2.96	...	7.0	3.20	...	10.5	3.36	...	8.7	3.40	...	7.1	3.29	...	7.4	3.52	...	9.1	3.78	...	9.6	3.68	...	8.7	3.46	...	7.9
4	2.46	...	5.4	2.07	...	5.6	2.58	...	6.4	2.19	...	5.0	2.87	...	5.5	2.33	...	5.8	2.33	...	7.5	2.30	...	8.0	2.14	...	6.1	2.48	...	6.6	2.07	...	6.8	2.18	...	5.9
5	3.88	...	8.1	3.78	...	8.7	4.34	...	12.8	3.45	...	8.2	3.38	...	8.3	3.54	...	11.2	3.67	...	9.5	3.17	...	8.3	3.58	...	10.0	3.62	...	8.1	2.89	...	10.8	3.88	...	10.8
6	3.62	...	7.9	3.44	...	9.5	3.91	...	10.3	3.33	...	7.9	3.56	...	11.8	3.24	...	8.9	3.37	...	9.2	3.47	...	8.5	3.36	...	8.2	4.08	...	13.1	2.97	...	10.6	3.85	...	9.4
7	3.32	2.5	8.5	3.30	3.1	9.2	3.77	3.5	8.8	3.32	2.8	7.5	3.37	4.9	7.6	3.30	5.0	10.9	3.30	4.0	10.8	3.32	2.5	8.1	3.10	5.8	9.6	3.25	3.9	7.5	3.56	2.8	7.9	3.78	2.7	7.1
8	3.52	...	7.2	3.62	...	10.3	4.20	...	11.0	3.64	...	11.5	3.58	...	9.1	3.72	...	12.3	4.12	...	10.4	4.25	...	11.1	3.62	...	9.0	4.01	...	12.3	4.16	...	9.0	3.62	...	9.0
9	3.34	...	7.1	3.47	...	9.8	3.80	...	9.0	3.22	...	7.1	3.19	...	7.4	3.28	...	8.0	3.56	...	12.6	3.55	...	9.5	3.61	...	9.5	3.40	...	11.5	3.71	...	6.9	3.52	...	7.8
10	2.98	1.8	5.8	2.74	1.6	5.9	3.16	2.2	8.1	2.93	2.4	7.4	3.11	3.0	6.8	3.23	2.7	8.8	3.63	3.7	10.3	3.68	3.3	9.9	3.34	4.1	11.0	3.53	1.8	11.6	3.28	3.7	7.6	3.03	2.8	7.2
11	2.68	...	9.7	2.42	...	7.7	2.68	...	8.2	2.55	...	6.0	3.19	...	7.4	3.54	...	7.8	3.47	...	9.7	3.45	...	9.8	3.26	...	7.8	3.17	...	9.3	2.86	...	6.6	2.63	...	5.0
12	3.09	...	6.1	2.80	...	6.4	3.37	...	7.2	2.87	...	7.2	3.06	...	6.2	3.48	...	11.5	2.77	...	8.7	3.70	...	10.8	3.74	...	8.1	3.20	...	5.6	3.36	...	9.2	3.02	...	6.8
13	4.27	...	8.5	4.18	...	9.0	5.92	...	14.5	5.94	...	33.4	6.17	...	12.5	9.08	...	18.5	10.34	...	23.9	5.62	...	15.4	5.42	...	15.2	7.45	...	15.4	6.62	...	17.5	5.22	...	15.9
14	2.21	...	5.6	2.25	...	5.4	2.42	...	6.0	2.73	...	6.1	2.98	...	6.8	3.33	...	9.8	4.07	...	7.2	2.67	...	7.1	2.89	...	8.0	2.94	...	6.1	2.76	...	6.0	2.46	...	5.2
15	1.82	1.0	7.5	1.65	1.6	4.2	2.05	2.0	4.1	2.12	1.3	5.8	2.96	1.9	9.8	3.47	3.1	9.9	3.80	3.3	8.4	3.40	2.5	9.6	2.60	2.8	9.8	3.20	2.2	8.2	2.83	4.5	6.2	1.90	1.4	4.8
16	2.24	...	5.0	2.21	...	4.8	2.35	...	6.4	2.29	...	5.1	2.69	...	6.9	3.20	...	7.9	3.46	...	8.0	3.52	...	7.0	3.01	...	6.2	2.62	...	5.8	2.83	7.0	11.4	2.34	...	5.8
17	2.72	...	4.7	2.63	...	6.0	2.92	...	6.3	2.54	...	6.2	2.92	...	5.7	3.29	...	9.0	3.40	4.0	10.6	3.49	...	8.4	3.34	...	8.2	2.89	...	5.7	3.11	4.5	10.9	2.50	...	5.2
18	2.99	...	6.2	2.79	...	5.9	3.10	...	7.0	2.77	...	7.1	3.21	...	7.2	3.38	...	7.2	3.95	...	11.1	3.30	...	8.8	3.41	...	7.9	3.07	...	12.9	3.28	...	6.5	2.82	...	6.2
19	2.58	...	7.2	2.38	...	6.2	2.80	...	9.9	2.88	...	5.8	3.07	...	6.2	3.33	...	9.1	4.28	...	10.8	4.26	...	14.8	3.40	...	7.0	3.06	...	12.9	3.00	...	6.5	2.94	...	6.6
20	3.31	...	7.9	3.50	...	6.7	3.62	...	8.5	3.19	...	7.8	3.72	...	8.9	3.58	...	10.0	4.32	...	15.0	4.24	...	11.7	3.45	...	11.4	3.86	...	13.5	3.53	...	8.1	3.38	...	8.8
21	3.38	...	6.7	3.26	...	8.1	3.58	...	7.1	3.23	...	8.8	3.75	...	8.7	3.63	...	11.7	4.37	...	14.5	4.28	...	12.1	3.60	...	11.8	3.68	...	11.4	3.64	...	7.5	3.34	...	7.8
22	3.10	...	7.8	3.11	...	9.9	3.50	...	8.0	3.48	...	10.2	3.37	...	7.6	3.29	...	10.4	3.86	...	10.2	4.08	...	12.8	3.31	...	11.0	3.45	...	7.9	3.60	...	8.0	3.33	...	8.6
23	3.65	3.2	8.8	3.46	4.4	10.0	3.95	3.0	11.7	3.74	3.2	10.8	3.55	3.3	10.4	3.12	5.8	8.1	3.59	6.0	12.4	4.00	5.0	12.1	3.84	4.8	11.9	3.48	4.9	8.8	3.96	5.4	11.6	3.71	2.9	9.0
24	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
25	3.88	...	8.9	3.63	...	9.4	3.89	...	8.4	3.06	...	9.3	3.44	...	7.7	3.09	...	7.5	3.46	...	12.4	4.18	...	15.0	3.49	...	9.8	3.48	...	8.1	3.89	...	10.4	3.85	...	8.2
26	3.58	...	7.9	3.65	...	7.5	3.62	...	7.0	3.45	...	8.8	3.25	...	7.8	3.22	...	8.5	3.49	...	11.7	3.44	...	7.3	3.42	...	8.4	3.17	...	10.2	3.37	...	9.2	3.41	...	7.7
27	3.32	...	9.1	2.91	...	8.0	3.60	...	7.9	3.70	...	10.6	3.50	...	10.0	3.29	...	10.6	3.66	...	11.7	4.11	...	14.0	3.46	...	9.9	3.57	...	11.1	3.80	...	10.8	3.25	...	8.8
28	3.84	...	7.8	3.76	...	8.6	4.14	...	10.9	3.84	...	10.2	3.54	...	8.1	3.36	...	11.2	3.89	...	13.3	4.22	...	12.4	3.58	...	9.9	3.76	...	10.1	4.08	...	8.5	3.65	...	8.6
29	4.02	...	10.7	3.86	...	8.3	4.27	...	9.8	3.96	...	9.3	3.90	...	9.4	3.11	...	8.0	3.34	...	12.0	4.17	...	18.7	3.40	...	12.1	3.85	...	10.1	4.12	...	9.7	4.09	...	10.0
30	3.79	...	8.0	3.16	...	7.0	3.71	...	8.9	3.31	...	8.5	3.18	...	7.9	2.79	...	7.7	2.71	...	5.8	3.24	...	11.0	2.62	...	8.3	3.58	...	10.0	3.26	...	7.8	3.88	...	6.7
31	3.80	...	8.6	3.70	...	9.7	3.87	...	8.0	3.57	...	8.0	3.51	...	6.8	2.62	...	12.9	3.04	...	7.9	3.46	...	9.8	2.68	...	7.8	3.57	...	7.4	2.62	...	9.4	3.75	...	8.7
32	4.12	2.1	8.8	3.81	2.4	11.3	4.13	1.6	9.7	3.80	2.2	9.4	3.64	2.5	10.6	3.28	3.0	9.8	3.45	3.8	10.3	4.02	3.8	8.8	3.34	4.8	9.8	3.57	3.1	9.3	3.93	3.3	9.1	3.97	2.7	9.4
33	3.88	...	9.0	3.63	...	9.5	4.30	...	13.2	3.74	...	9.0	3.28	...	7.4	3.04	...	8.3	3.72	...	10.0	4.35	...	12.3	3.22	...	8.0	3.67	...	9.4	3.78	...	9.1	3.78	...	8.1
34	3.85	...	8.6	3.60	...	12.0	4.12	...	11.0	3.63	...	10.8	3.39	...	8.1	3.10	...	7.7	3.60	...	7.1	4.22	...	16.4	3.36	...	10.4	3.71	...	8.8	3.62	...	7.8	3.60	...	7.4
35	3.96	...	9.2	3.88	...	9.0	4.42	...	9.5	3.52	...	12.4	3.79	...	7.7	3.38	...	11.4	4.11	...	13.4	4.50	...	10.3	3.58	...	11.6	3.97	...	15.8	3.82	...	8.1	3.81	...	10.8
36	3.88	...	9.3	3.81	...	9.0	4.00	...	8.8	3.38	...	10.1	3.53	...	7.4	3.08	...	10.5	4.24	...	15.3	4.40	...	9.6	3.46	...	11.6	3.46	...	9.2	3.58	...	8.3	3.82	...	8.9
37	3.59	...	9.8	3.69	...	9.3	4.04	...	9.6	3.52	...	12.3	4.23	...	18.0	4.15	...	12.0	4.52	...	17.0	4.68	...	26.9	4.12	...	11.2	4.42	...	14.7	4.12	...	9.3	3.76	...	9.1
38	3.81	...	8.6	3.97	...	7.8	4.00	...	10.4	3.56	...	7.9	3.80	...	9.3	3.19	...	8.9	4.29	...	17.1	4.39	...	12.0	3.56	...	13.4	3.73	...	10.1	3.52	...	8.3	3.74	...	7.6
39	3.58	...	6.1	3.70	...	7.8	3.74	...	7.9	3.29	...	7.0	3.18	...	9.1	3.31	...	7.9	4.33	...	9.6	4.46	...	10.4	3.40	...	14.8	3.57	...	11.5	3.12	...	9.8	3.41	...	6.7
39-A	3.78	2.5	7.4	3.31	3.2	7.8	3.39	3.6	7.9	3.28	3.7	11.2	3.44	4.2	9.1	3.43	3.9	12.8	4.13	3.8	9.6	4.28	5.0	14.2	3.38	6.2	14.5	3.45	9.4	11.5	3.33	3.6	9.8	3.31	2.9	7.1
40	4.45	...	11.8	3.95	...	13.3	4.21	...	13.2	[illegible]	...	9.8	3.67	...	9.0	3.92	...	12.2	5.05	...	22.7	4.35	...	16.5	3.74	...	14.7	4.15	...	14.1	3.22	...	8.8	3.90	...	10.1
41	3.82	...	9.9	3.72	...	7.3	3.62	...	12.0	[illegible]	...	10.5	4.37	...	11.7	4.37	...	7.4	3.56	...	13.0	4.13	...	11.7	3.67	...	13.5	3.73	...	10.2	3.22	...	10.0	3.38	...	7.0
42	4.02	...	9.8	3.99	...	8.4	4.30	...	8.3	3.76	...	8.5	3.99	...	8.8	3.56	...	11.8	4.09	...	13.5	4.59	...	10.3	4.22	...	14.4	4.06	...	9.5	3.78	...	8.2	4.08	...	9.8
43	2.54	1.6	7.3	2.42	1.9	5.2	2.74	2.3	7.4	2.70	2.0	7.0	3.35	2.2	8.5	3.32	2.9	8.7	3.91	3.1	9.4	3.73	4.2	10.6	3.32	3.3	8.9	3.25	4.7	13.5	2.93	7.5	7.2	2.60	2.4	4.2
44	2.00	...	5.2	1.64	...	6.2	2.17	...	7.1	1.97	...	5.6	2.56	...	7.1	3.12	...	7.6	2.54	...	9.2	3.19	...	7.2	2.06	...	7.6	2.81	...	8.3	2.57	...	8.8	[illegible]	...	7.2
45	2.72	1.2	6.1	2.43	2.1	5.4	2.80	1.2	4.7	2.26	1.6	3.9	3.33	2.4	7.2	3.06	3.5	6.8	3.60	2.3	8.3	3.40	2.9	5.9	2.62	2.7	6.1	3.10	2.4	5.9	3.11	1.3	6.5	2.50	1.2	6.3
46	3.01	2.4	6.5	2.62	1.6	4.9	2.78	2.7	7.1	2.44	2.2	7.0	3.04	2.4	7.4	3.25	2.8	9.8	3.21	3.7	7.7	2.76	3.8	6.7	2.94	1.9	6.9	3.57	3.0	7.5	2.34	1.9	8.4	2.50	2.6	10.5
47	2.84	1.7	4.7	2.45	1.9	4.5	2.97	2.0	4.6	2.68	1.5	4.3	2.71	2.2	4.9	3.46	2.7	6.7	3.31	2.6	7.4	3.03	2.0	8.1	2.93	2.5	6.6	3.20	1.9	6.4	2.14	1.3	4.4	2.92	2.2	5.1
48	2.60	2.1	8.0	2.39	2.3	5.4	2.58	3.0	7.0	2.46	1.8	6.1	3.08	2.9	6.9	3.09	2.3	8.3	3.16	3.6	6.4	2.85	4.2	7.3	2.77	2.1	7.4	2.92	2.8	8.7	2.00	2.3	5.8	1.94	2.2	6.3
49	1.96	...	5.7	1.95	...	4.1	2.03	...	4.5	1.87	...	4.2	2.80	...	7.3	2.31	...	7.2	2.81	...	7.0	2.38	...	7.2	2.83	...	7.2	2.22	...	5.2	2.23	...	4.8	2.06	...	6.0
50	3.09	2.0	6.4	2.84	1.8	6.7	2.78	1.5	5.5	2.52	1.9	4.7	3.03	2.1	7.3	2.88	3.3	9.5	3.10	3.6	8.3	3.01	4.2	10.6	3.17	2.4	7.7	3.40	3.1	7.6	3.13	3.3	6.7	3.27	2.6	5.8
51	3.24	...	7.1	2.92	...	5.6	3.54	...	6.5	3.07	...	5.3	3.44	...	6.8	3.74	...	7.2	4.78	...	11.2	3.82	...	9.5	3.59	...	8.8	3.66	...	7.3	2.58	...	7.2	2.94	...	7.0
52	3.06	...	4.9	2.34	...	4.4	3.04	...	4.7	3.05	...	6.1	3.90	...	11.0	4.03	...	10.0	4.78	...	10.9	4.02	...	6.6	3.10	...	5.2	3.28	...	9.7	2.54	...	6.2	2.66	...	5.8
53	2.08	2.0	8.0	1.86	1.6	4.6	2.34	2.5	4.8	2.36	2.0	4.8	3.42	2.5	7.3	3.62	3.0	7.5	3.44	2.6	6.7	3.17	2.7	8.4	3.08	4.2	7.5	2.91	3.2	5.9	2.36	2.8	7.0	2.37	3.1	5.7
54	2.77	...	5.5	2.62	...	6.7	2.86	...	6.0	2.79	...	7.1	3.06	...	7.8	4.22	...	10.0	4.61	...	9.2	4.37	...	9.7	3.01	...	8.1	3.44	...	8.3	3.02	...	5.4	2.58	...	6.0
55	2.32	1.9	5.1	2.23	1.9	4.3	2.67	1.9	4.7	2.54	1.7	4.2	3.25	3.0	7.0	3.34	2.6	6.8	3.52	4.2	8.1	3.63	3.2	7.7	3.19	4.5	6.9	2.95	2.9	5.9	2.22	2.7	5.9	2.29	3.4	5.4
56	2.70	1.7	4.7	2.88	3.6	6.3	3.22	2.7	5.7	2.79	1.8	5.2	2.98	1.7	8.6	3.61	3.0	7.7	3.84	3.0	6.8	3.61	2.9	8.2	3.29	5.1	7.8	3.09	3.7	9.2	2.49	3.2	5.6	2.90	2.1	5.6
57	3.41	3.3	6.7	3.38	3.9	6.9	3.45	2.8	9.1	2.91	2.8	9.8	3.20	3.2	9.5	3.30	3.4	8.0	4.33	3.2	10.3	4.61	5.9	12.1	3.38	5.6	12.1	3.10	3.7	6.7	3.06	2.6	7.8	3.04	3.8	7.3
58	3.29	...	7.8	3.18	...	6.9	3.50	...	9.1	3.37	...	9.8	3.68	...	9.5	3.65	...	11.0	4.18	...	11.8	4.49	...	15.8	3.48	...	12.1	3.19	...	10.0	3.28	...	8.0	3.46	...	7.4
59	3.56	2.9	7.3	3.90	3.9	7.3	4.07	2.9	8.3	3.78	4.2	8.8	4.45	3.6	13.8	4.19	6.1	9.8	4.83	4.5	14.6	4.68	6.6	12.3	3.92	5.0	12.8	3.74	5.8	8.7	3.08	1.6	9.9	4.07	3.8	8.6
60	3.50	2.8	6.5	3.50	3.3	6.7	3.52	2.4	7.2	3.34	2.3	6.5	3.69	2.9	13.3	3.45	2.7	8.2	4.38	5.8	12.4	4.23	5.4	11.8	3.27	3.8	8.6	2.61	2.8	9.5	2.88	2.0	8.9	3.42	2.8	7.1
61	3.38	2.2	6.3	3.47	3.0	6.4	3.84	2.5	6.6	3.37	2.2	6.3	4.03	3.0	10.6	4.06	2.2	10.0	4.27	3.2	11.4	4.61	4.8	12.9	3.18	2.0	10.2	3.52	3.2	8.2	2.88	2.4	10.0	3.62	1.9	7.5
62	3.07	2.8	10.1	2.82	2.6	5.5	3.04	2.3	5.8	3.10	2.0	5.9	3.78	3.2	10.0	3.91	1.9	11.2	3.79	2.3	13.6	4.18	4.7	12.2	3.36	5.1	12.4	2.97	3.3	6.4	2.25	1.9	6.8	2.14	1.9	9.0
63	2.60	1.4	6.3	2.11	1.6	4.6	2.70	1.7	5.8	2.87	3.6	6.4	4.02	2.9	9.5	3.92	2.0	9.8	3.27	2.6	9.0	3.39	4.2	8.4	2.78	2.6	6.6	2.49	2.3	7.9	1.91	1.6	6.4	2.35	1.6	5.4
64	4.29	1.9	7.8	3.75	1.6	10.7	4.04	2.8	10.0	3.96	1.5	7.6	4.59	2.8	12.2	5.19	4.5	11.0	4.40	2.8	8.9	4.55	2.6	8.3	3.60	2.6	10.9	2.99	2.3	8.5	2.92	2.0	6.1	3.65	3.1	6.1
65	3.88	...	7.8	3.16	...	7.7	3.62	...	6.7	3.32	...	8.9	3.88	...	7.8	4.20	...	7.1	4.18	...	8.1	3.72	...	8.8	3.87	...	8.7	2.88	...	7.7	2.88	...	6.4	3.38	...	6.8
66	3.02	1.9	7.7	2.80	1.8	9.2	3.45	2.2	7.6	3.03	2.8	5.9	3.49	1.9	10.0	4.77	2.8	9.8	4.45	3.1	11.6	3.80	2.2	7.4	3.46	3.2	10.2	3.11	1.9	8.9	3.01	1.7	5.5	3.22	1.9	5.9
67	2.96	1.9	6.2	2.69	2.3	8.5	2.67	3.1	7.4	2.65	2.4	5.3	3.56	4.7	8.0	3.57	2.6	8.8	3.06	3.2	7.0	3.37	5.8	9.3	3.42	3.4	8.4	3.81	3.2	6.3	3.27	2.1	6.5	2.94	1.9	6.4
68	3.02	2.3	6.2	2.57	2.0	6.8	3.04	2.0	8.8	2.92	2.6	8.1	3.12	3.0	6.4	3.68	3.2	6.9	4.21	3.8	9.5	3.22	3.6	7.3	2.62	4.1	7.4	2.55	2.0	6.3	2.34	2.1	5.1	3.41	2.4	6.3
69	2.73	2.3	6.2	2.42	2.0	6.8	2.91	2.0	8.8	3.01	2.6	9.3	3.22	3.0	6.6	3.70	3.2	7.6	3.89	3.8	9.5	3.27	3.6	8.3	2.88	4.1	8.2	2.70	2.0	6.2	2.45	3.1	5.2	2.88	2.4	6.2
70	4.76	2.1	8.9	3.98	1.8	7.7	4.34	2.7	8.8	4.64	3.1	10.0	5.02	2.3	14.3	5.14	2.6	10.3	4.44	2.5	10.5	4.72	4.4	11.2	3.67	3.2	11.4	3.12	1.8	8.0	3.46	3.0	8.4	3.96	2.1	7.3
71	3.78	...	7.7	3.08	...	7.1	3.66	...	7.2	3.28	...	7.2	3.57	...	8.1	4.56	...	7.2	4.98	...	11.3	4.24	...	10.1	2.06	...	7.7	3.06	...	8.2	2.92	...	7.9	3.14	...	8.7
72	3.60	3.0	8.2	3.00	2.9	7.4	3.45	2.4	6.9	3.20	3.4	6.5	3.27	3.0	6.6	4.12	3.2	8.2	4.42	3.8	11.5	3.64	3.6	7.4	2.68	2.9	8.4	2.47	3.1	6.5	2.08	9.2	6.6	2.92	2.4	5.6
73	4.18	...	7.5	3.78	...	6.5	4.36	...	10.0	3.94	...	7.8	4.31	...	9.1	4.62	...	8.2	4.95	...	15.1	3.99	...	9.8	3.00	...	5.8	2.99	...	7.8	3.28	...	9.7	3.22	...	7.3
74	4.31	1.8	8.9	3.14	1.6	5.7	4.10	2.4	7.1	3.84	2.2	7.0	4.15	2.7	8.9	5.22	3.0	8.0	5.80	4.3	11.1	4.10	3.4	10.4	3.00	2.7	7.1	2.97	2.2	6.8	2.77	3.1	5.9	3.72	2.8	6.9
75	3.22	3.0	7.1	2.97	2.3	6.8	3.43	2.8	8.8	3.31	3.2	9.1	3.61	3.8	10.7	3.80	4.2	10.9	4.38	5.8	10.6	4.15	7.8	14.4	3.19	5.7	10.8	3.01	4.0	10.6	2.51	2.8	7.2	3.02	2.7	7.6
76	3.40	3.0	7.1	3.20	2.3	6.8	3.82	2.8	8.8	3.22	3.2	9.1	3.60	3.8	10.7	4.18	4.2	10.9	4.74	5.8	10.6	4.13	5.0	14.4	3.37	5.7	10.8	2.88	4.0	8.7	2.49	2.8	7.2	3.22	3.7	7.6
77	2.95	3.2	7.1	3.00	3.5	7.1	3.62	3.5	9.1	3.20	3.7	9.1	3.43	3.0	9.1	3.54	4.5	9.2	3.98	4.0	11.0	4.00	4.4	12.3	3.28	6.1	10.7	3.01	5.2	7.8	2.92	3.6	7.9	3.25	3.2	8.8
78	3.38	3.2	6.8	3.30	3.5	7.1	3.70	3.5	7.9	3.35	3.7	8.7	3.46	3.0	7.3	2.75	4.5	8.2	4.89	4.0	11.0	4.39	4.4	13.3	3.52	6.1	10.5	2.94	5.3	6.9	2.73	3.3	6.9	3.20	3.2	7.1
79	3.54	2.6	7.5	3.52	2.7	6.2	4.25	2.5	8.9	3.70	3.7	10.1	3.98	4.3	10.7	3.94	3.8	8.6	5.15	5.9	15.2	5.74	4.6	14.1	3.88	4.0	10.1	3.72	5.4	7.6	3.83	2.1	8.4	3.55	2.8	7.5
80	3.76	...	8.1	3.62	...	6.8	3.87	...	10.0	3.74	...	11.4	3.97	...	9.0	3.76	...	11.5	4.81	...	19.1	5.07	...	22.5	3.76	...	17.7	3.87	...	13.8	3.47	...	8.9	3.58	...	7.6
81	3.46	3.0	7.2	3.28	2.7	7.4	3.69	2.6	7.0	3.10	3.1	6.0	3.06	3.1	5.8	3.08	2.9	6.1	3.61	5.4	10.1	4.33	9.0	14.9	2.89	3.9	6.2	3.14	9.2	12.1	2.97	3.7	6.8	3.90	3.5	8.2
82	3.08	...	11.0	2.90	...	8.2	3.57	...	10.8	3.12	...	9.8	3.67	...	7.7	3.68	...	11.1	4.68	...	18.0	4.78	...	14.2	3.67	...	16.4	3.02	...	8.8	2.90	...	10.6	3.47	...	9.0
83	3.14	2.2	6.9	3.43	3.4	7.3	3.95	2.8	9.1	3.37	5.1	11.9	3.85	2.8	8.6	4.22	4.0	9.8	5.76	3.6	11.9	5.33	6.0	11.2	3.85	4.2	11.9	3.20	6.3	10.0	2.40	6.5	6.7	3.41	2.2	6.1
84	3.14	2.7	5.8	3.15	3.2	8.9	3.70	3.6	8.8	3.44	3.8	8.9	3.78	2.4	9.1	3.80	3.3	8.7	4.58	7.3	14.0	4.50	3.4	10.4	3.19	4.0	10.4	2.81	2.9	8.5	2.22	2.8	6.7	3.12	2.5	7.1
85	3.28	...	6.2	2.92	...	7.1	3.41	...	8.8	3.40	...	9.0	4.11	...	11.5	5.09	...	15.8	4.44	...	12.0	4.86	...	14.4	3.98	...	13.1	3.62	...	10.8	2.60	...	6.8	3.80	...	8.3
86	3.82	...	7.2	3.97	...	5.8	3.32	...	7.1	2.92	...	7.9	3.68	...	7.4	4.15	...	7.6	4.01	...	8.7	3.82	...	10.6	2.47	...	15.9	2.90	...	7.8	2.62	...	9.2	2.82	...	8.1
87	3.47	...	7.3	3.22	...	7.8	3.62	...	7.4	3.11	...	8.3	3.71	...	9.4	3.97	...	8.8	4.14	...	10.9	4.07	...	12.4	3.54	...	10.7	3.15	...	8.9	2.62	...	9.6	3.11	...	8.7
88	3.37	...	7.0	3.39	...	6.4	3.67	...	9.7	3.14	...	8.0	4.00	...	8.4	4.11	...	8.0	4.58	...	9.4	4.67	...	11.9	3.82	...	6.8	2.74	...	7.1	2.37	...	7.6	3.22	...	6.7
89	3.04	1.7	7.1	3.14	1.9	9.4	3.59	2.6	8.0	3.10	2.1	9.1	3.61	2.8	9.0	4.41	2.9	8.4	4.20	3.1	8.6	4.31	3.2	13.1	3.33	3.8	12.2	2.70	4.2	6.7	2.19	1.6	5.8	2.78	2.0	7.6
90	2.60	2.9	6.7	2.62	1.7	7.0	3.64	3.2	7.1	3.09	2.4	5.4	3.48	3.3	7.7	4.29	2.7	7.7	4.70	3.3	11.7	4.89	5.8	9.1	2.99	3.0	6.2	3.04	7.9	11.3	1.72	1.9	4.1	3.09	2.6	6.5
91	5.66	...	14.8	5.97	...	15.1	6.47	...	15.3	4.76	...	15.4	3.89	...	10.5	5.34	...	9.3	5.97	...	11.3	5.35	...	12.2	3.44	...	7.4	3.41	...	7.4	3.81	...	9.3	5.20	...	12.8
92	6.78	...	16.6 12.9	7.10	...	14.0 13.1	7.57	...	16.7 16.2	6.12	...	14.2 11.4	5.68	...	12.6 11.0	8.32	...	19.9 17.8	9.43	...	26.4 17.1	8.48	...	26.4 17.1	6.58	...	22.5 15.6	5.72	...	18.5 18.4	4.78	...	12.7 12.1	7.67	...	15.2 14.3
93	3.98	...	8.9	4.28	...	7.9	4.27	...	11.1	3.28	...	7.2	3.76	...	11.0	4.31	...	11.0	5.29	5.0	16.5	5.29	...	14.6	2.97	...	8.2	3.07	...	8.3	2.88	...	6.0	2.77	...	6.6
94	3.51	2.4	7.4	4.01	3.0	7.5	3.88	2.8	7.3	3.35	3.4	7.9	6.07	4.0	9.9	4.82	5.2	10.4	5.56	3.7	11.2	5.51	5.7	13.6	3.49	3.1	10.9	2.90	4.2	10.2	3.22	3.9	6.7	3.40	2.4	7.8
95	4.40	...	13.9	4.16	...	8.1	4.50	...	12.8	3.84	...	11.1	3.72	...	7.8	4.61	...	11.9	5.35	...	11.7	5.72	...	16.3	5.16	...	15.4	5.06	...	12.4	3.56	...	13.0	4.55	...	13.4
96	3.37	4.6	8.4	3.41	3.2	6.6	3.38	5.3	7.1	2.78	2.7	6.6	3.61	2.9	8.6	5.36	7.0	12.4	6.73	7.3	21.1	6.74	8.0	14.3	4.94	7.3	20.1	3.48	4.5	10.1	2.11	2.9	5.5	2.96	3.0	7.1
96-A	3.28	...	7.3	4.94	...	9.4	3.78	...	10.6	3.32	...	8.3	3.53	...	9.0	5.86	...	14.1	5.76	13.2	24.9	6.10	...	15.4	4.01	...	9.2	2.98	...	6.6	2.10	...	5.1	3.18	...	8.1
97	2.98	4.0	7.8	3.05	3.2	10.4	3.26	3.1	12.1	2.41	8.3	15.0	3.38	5.9	8.9	5.08	5.9	16.5	6.25	7.6	13.7	6.52	5.9	19.2	5.10	7.0	16.1	3.72	9.5	14.3	2.54	5.8	8.3	3.20	3.5	9.9
98	3.18	...	7.6	4.15	...	9.1	3.37	...	8.3	2.99	...	7.5	3.32	...	10.1	4.54	...	10.0	5.40	...	12.2	5.77	...	9.8	3.65	...	9.7	2.48	...	7.4	2.18	...	5.8	2.98	...	7.3
99	3.38	...	10.7	3.97	...	9.3	3.70	...	9.6	3.08	...	7.7	3.21	...	9.1	4.48	...	9.5	5.22	...	14.9	5.46	...	14.7	3.66	...	8.3	2.62	...	7.2	2.35	...	8.5	3.40	...	7.8
100	3.65	...	10.6	5.25	...	11.6	5.29	...	11.6	3.65	...	8.2	3.56	...	12.8	5.09	...	11.0	5.72	...	12.1	5.06	...	19.5	3.79	...	10.3	3.20	...	11.4	3.10	...	9.5	4.71	...	9.9
100-A	3.92	...	7.2	5.12	...	10.0	4.60	...	10.0	3.18	...	6.2	3.04	...	10.4	3.75	...	12.4	5.04	...	11.3	5.33	11.0	18.4	3.06	...	8.2	2.78	...	5.9	2.70	...	7.8	4.87	...	8.8
101	3.78	...	8.7	5.18	...	10.0	4.53	...	11.9	3.28	...	7.6	2.96	...	8.0	4.56	...	9.6	5.00	...	14.0	5.25	...	12.5	3.48	...	8.9	2.48	...	7.5	2.67	...	7.8	3.39	...	8.1
102	6.02	4.0	15.8	4.79	3.8	10.4	5.42	7.4	11.9	3.06	2.8	8.2	3.54	4.9	9.8	2.75	3.9	11.2	4.74	5.6	14.1	4.49	4.2	10.0	3.10	5.9	14.5	2.62	3.5	6.4	3.08	3.2	8.2	4.61	5.7	12.9
103	3.06	3.5	7.9	4.44	4.8	8.3	5.02	6.4	12.9	3.73	3.9	11.4	2.89	3.7	8.2	3.90	6.8	9.1	4.85	8.4	13.4	4.34	2.8	10.1	3.02	4.4	10.6	2.27	2.6	8.5	2.77	3.8	8.2	3.99	2.8	7.4
104	2.84	3.5	8.8	3.24	3.0	9.7	3.84	3.3	10.2	2.85	6.1	8.8	3.06	5.6	11.1	5.65	6.1	18.8	6.48	6.4	14.3	7.45	8.6	22.8	5.42	10.1	22.9	3.10	4.8	9.4	2.17	4.1	6.2	2.93	2.9	8.0
104-A	3.87	...	8.5	2.99	...	10.0	[illegible]	...	7.3	2.46	...	7.3	3.92	...	10.6	6.52	...	15.0	6.22	...	10.4	5.79	18.0	24.2	5.14	...	22.4	2.51	...	7.1	1.88	...	6.4	3.62	...	8.8
105	3.82	3.1	9.1	3.07	4.2	9.2	3.06	4.8	8.9	2.50	4.8	7.9	4.12	9.1	14.8	5.46	7.7	12.8	6.44	4.9	15.0	6.02	6.2	11.0	7.40	9.9	21.8	4.58	5.1	16.2	2.12	3.7	6.1	3.08	5.0	7.8
106	2.62	...	8.2	3.25	...	21.5	2.82	...	8.8	2.61	...	8.1	3.63	...	10.7	5.29	...	14.2	5.28	...	12.2	5.78	...	13.0	6.48	...	23.9	3.27	...	24.3	4.98	...	22.7	2.68	...	6.8
107	3.52	6.4	9.8	2.56	3.6	6.4	3.11	4.0	9.2	2.42	4.2	8.5	5.00	6.8	13.5	6.82	9.8	17.4	5.33	3.4	9.8	5.86	4.9	12.1	9.06	6.7	18.1	10.26	13.2	21.4	3.02	5.1	9.0	2.60	5.7	15.2
108	3.91	6.7	8.9	2.08	2.5	5.9	2.52	9.0	9.7	3.30	3.2	10.7	6.75	6.2	16.7	6.92	4.0	21.7	6.37	4.9	15.2	6.49	6.1	14.5	8.63	8.0	25.1	9.12	9.5	27.9	3.06	15.1	17.7	1.99	2.5	12.1
109	1.90	4.0	7.6 5.4	1.36	3.0	7.2 4.7	1.48	4.5	10.2 6.9	1.51	3.2	8.0 4.4	3.72	5.8	13.0 11.5	4.04	5.5	17.1 15.9	3.36	7.5	11.0 10.9	4.06	4.0	14.6 13.0	6.96	11.9	17.3 16.1	5.10	11.2	19.5 17.0	2.08	8.9	10.8 6.0	1.76	3.9	10.0 8.4
110	2.48	2.6	6.7	2.51	4.1	6.9	2.59	2.1	7.7	1.89	2.9	8.8	2.92	2.4	9.4	7.13	6.5	12.6	8.94	5.2	24.5	8.52	5.0	23.4	7.08	8.6	18.9	2.72	2.9	10.3	1.77	4.2	4.8	2.18	3.9	7.4
111	3.75	3.1	11.9	3.51	3.6	10.4	3.43	3.8	10.6	2.95	4.2	8.5	3.82	4.7	9.3	4.70	8.4	14.9	7.68	7.1	17.3	7.65	7.0	14.1	8.50	10.1	27.7	3.42	4.8	9.1	2.80	4.8	6.8	5.18	6.0	15.9
112	4.07	3.5	10.0	4.38	5.0	12.5	4.75	8.8	13.4	3.90	8.9	8.9	3.38	8.6	9.2	4.63	10.7	14.1	6.72	5.0	17.9	7.84	9.6	18.6	5.38	7.7	17.9	4.13	7.3	14.7	3.67	7.7	18.5	4.60	4.8	11.1
113	4.82	...	11.9	5.22	...	11.1	6.28	...	14.6	4.74	...	17.3	4.32	...	12.1	5.48	...	26.7	6.92	...	20.5	6.82	...	15.2	5.08	...	16.4	3.49	...	11.2	3.75	...	10.2	4.90	...	20.0
114	5.20	10.0	17.8	5.54	5.2	11.8	6.19	6.1	16.5	4.63	6.0	15.9	3.88	3.6	10.2	3.91	3.8	11.1	4.78	5.7	13.4	4.03	3.4	8.5	3.08	6.1	12.0	3.34	3.5	10.3	3.38	6.6	8.7	4.84	5.1	10.0
115	5.15	4.3	8.9	4.61	5.6	15.9	5.77	4.8	11.8	4.90	4.2	13.1	4.08	4.3	10.5	4.38	4.1	14.4	5.35	8.8	20.1	4.22	3.2	13.8	3.14	7.6	13.2	3.38	5.6	10.8	2.98	2.8	6.4	5.22	4.8	10.6
116	4.92	...	8.3	4.90	...	10.6	5.77	...	16.1	4.74	...	16.1	4.08	...	10.3	4.48	...	13.9	4.84	...	14.6	3.98	...	10.6	3.11	...	7.9	2.54	...	11.0	3.18	...	6.8	4.64	...	13.0
117	5.08	4.1	9.8	5.30	5.0	13.2	4.96	5.8	10.6	4.82	9.5	14.9	4.44	4.9	12.1	4.82	3.7	20.1	5.35	6.2	14.2	4.87	4.9	12.1	3.38	6.9	11.7	2.18	3.0	8.6	2.96	3.2	6.8	5.08	7.1	12.8
118	5.18	7.0	13.8	4.76	3.6	14.0	5.38	7.1	14.5	5.28	4.6	22.3	4.22	4.9	13.2	3.91	3.4	11.3	4.57	5.0	10.9	3.50	4.0	11.1	3.24	5.2	10.5	2.76	6.4	17.3	4.12	4.8	14.1	5.16	5.8	13.8

TABLE 1—

Station No. (1)	Station (2)	Elevation above sea, in feet (3)	Years of Record. For day or less. (4)	Years of Record. For month and year. (5)	Years of Record. Last year included. (6)	Maximum Observed Precipitation. Number of Minutes. 5 (7)	Maximum Observed Precipitation. Number of Minutes. 15 (8)	Maximum Observed Precipitation. Number of Minutes. 60. (9)	Maximum Observed Precipitation. Day of 24 hours. (10)	Maximum Observed Precipitation. Month. (11)	Annual Precipitation. Minimum. (12)	Annual Precipitation. Mean. (13)	Annual Precipitation. Maximum. (14)	Observed Precipitation, in Percentages of Mean Annual Rainfall. Mean daily intensity on days of rainfall. (15)	Maximum During Short Periods. Number of Minutes. 5. (16)	Maximum During Short Periods. Number of Minutes. 15. (17)	Maximum During Short Periods. Number of Minutes. 60. (18)	Mean for month of maximum record. (19)	Maximum. In 24 hours. (20)	Maximum. In one calendar month. (21)	Annual. Minimum. (22)	Annual. Mean. (23)	Annual. Maximum. (24)
119	New Orleans, La.	51	57	81	1927	....	....	....	14.0	22.7	31.1	56.8	85.7	0.8	....	....	....	10.1	24.6	40.0	55	100	151
120	Shreveport, La.	249	56	56	1927	0.59	1.20	3.18	8.6	16.5	23.1	44.3	66.6	1.0	1.3	2.7	7.1	6.9	19.4	37.3	52	100	150
121	Dallas, Tex.	466	16	51	1927	0.81	1.62	2.33	6.1	13.7	18.0	37.3	59.5	1.1	2.2	4.3	6.2	8.3	16.4	36.9	48	100	160
122	Fort Worth, Tex.	670	30	34	1927	0.56	1.40	2.75	8.8	17.6	17.9	31.6	48.4	1.3	1.8	4.4	8.7	13.0	27.9	55.7	57	100	153
123	Palestine, Tex.	510	41	41	1922 1920	0.59	1.56	4.46	6.3	17.2	24.0	40.7	61.2	1.1	1.4	3.8	10.9	11.1	15.5	42.2	59	100	150
124	Taylor, Tex.	583	22	25	1922	0.90	2.53	4.25	23.1	25.9	14.6	31.9	51.5	1.1	1.3	7.9	13.3	10.8	72.4	81.2	46	100	161
125	Austin, Tex.	593	....	65	1920	....	....	....	....	19.8	15.6	32.2	64.7	1.6	....	....	....	10.6	....	59.6	47	100	194
126	Houston, Tex.	138	32	45	1926	....	....	....	7.5	19.2	30.6	47.4	72.3	....	....	....	....	10.4	15.8	40.5	65	100	154
127	Galveston, Tex.	69	56	56	1926	....	....	5.32	14.4	26.0	21.4	47.1	78.4	1.0	....	....	11.8	11.5	30.6	55.2	45	100	167
128	San Antonio, Tex.	701	36	50	1920	....	....	....	7.1	11.7	10.1	27.7	50.3	1.2	....	....	....	11.9	25.6	42.2	36	100	182
129	Corpus Christi, Tex.	20	41	41	1927	....	....	....	8.3	13.8	6.4	24.9	48.2	1.2	....	....	....	16.4	33.8	55.5	26	100	194
130	Brownsville, Tex.	38	....	50	1920	....	....	....	....	30.6	12.2	26.4	60.1	1.5	....	....	....	20.2	....	116.0	22	100	228
131	Eagle Pass, Tex.	800	....	45	1920	....	....	....	....	13.5	7.0	20.8	44.4	2.3	....	....	....	12.4	....	65.6	34	100	213
132	El Paso, Tex.	3 762	48	48	1926	....	....	....	6.5	9.2	2.2	9.2	19.2	2.0	....	....	....	21.6	70.7	100.0	24	100	209
133	Abilene, Tex.	1 738	41	41	1926	0.67	1.51	3.47	6.8	15.7	10.9	24.4	41.6	0.7	2.7	6.2	14.2	9.6	27.9	64.4	45	100	171
134	Amarillo, Tex.	3 676	36	46	1927	0.53	1.19	1.95	4.0	9.8	11.1	22.0	39.7	1.4	2.4	5.4	8.9	13.7	18.2	44.6	50	100	181
135	Oklahoma, Okla.	1 247	....	30	1920	....	....	....	....	12.3	15.7	31.0	52.0	1.2	....	....	....	11.3	....	39.8	51	100	168
136	Fort Gibson, Okla.	556	....	50	1920	....	....	....	....	13.1	18.8	37.2	59.3	1.2	....	....	....	9.6	....	35.2	50	100	160
137	Bentonville, Ark.	1 303	17	17	1922	0.75	1.58	2.23	5.1	11.2	34.6	41.4	56.3	0.9	1.8	3.8	5.4	6.2	12.3	29.5	84	100	136
138	Fort Smith, Ark.	457	41	41	1922	0.57	1.14	2.06	5.1	15.0	19.8	38.3	64.6	1.1	1.5	3.0	5.4	5.8	13.3	29.2	52	100	169
139	Little Rock, Ark.	357	31	48	1927	0.60	1.12	2.42	9.6	16.9	31.6	48.2	75.5	0.8	1.2	2.3	5.0	5.0	19.9	35.0	66	100	156
140	Memphis, Tenn.	409	57	57	1927	0.78	1.08	1.97	9.7	18.2	33.4	48.5	73.5	0.9	1.6	2.2	4.1	7.9	20.0	37.5	69	100	152
141	Nashville, Tenn.	624	18	57	1927	0.75	1.26	2.09	6.0	14.5	22.7	47.2	67.2	0.8	1.6	2.7	4.4	9.9	12.7	30.7	69	100	142
142	Chattanooga, Tenn.	762	19	42	1920	0.60	1.31	2.55	7.6	15.3	32.7	51.3	70.6	0.8	1.2	2.6	5.0	9.3	14.8	29.8	64	100	138
143	Knoxville, Tenn.	996	50	53	1920	0.57	1.24	2.04	6.2	17.3	35.1	48.5	73.9	0.7	1.2	2.6	4.2	9.2	12.8	35.6	72	100	152
144	Lexington, Ky.	989	39	62	1920	0.34	0.91	2.37	5.4	11.1	28.7	44.5	63.3	0.7	0.8	2.0	5.3	8.7	12.1	25.0	64	100	142
145	Louisville, Ky.	525	50	57	1927	0.55	1.20	1.59	5.5	16.5	29.2	43.7	56.5	0.8	1.1	2.7	3.6	8.9	12.6	37.8	67	100	129
146	Evansville, Ind.	431	31	51	1927	0.53	1.19	2.79	6.9	14.6	29.0	43.0	70.6	0.9	1.2	2.8	6.5	7.4	16.1	34.0	67	100	164
147	Cairo, Ill.	358	50	57	1927	0.34	0.88	3.15	5.7	15.0	26.5	41.3	64.1	0.9	0.8	2.1	7.6	9.4	13.8	36.4	64	100	156
148	St. Louis, Mo.	567	57	91	1927	0.56	1.07	2.95	7.0	17.1	23.4	39.7	68.8	0.9	1.4	2.7	7.4	11.4	17.6	43.0	59	100	173
149	Hannibal, Mo.	534	25	48	1922	0.56	1.39	2.09	5.4	11.6	18.3	33.4	48.6	0.9	1.7	4.2	6.3	11.3	16.2	34.8	55	100	146
150	Columbia, Mo.	784	20	37	1922	0.70	1.22	2.73	6.6	11.5	21.3	37.6	54.6	0.9	1.9	3.2	7.3	12.1	17.6	30.6	57	100	145
151	Springfield, Mo.	1 302	40	50	1927	....	....	....	6.5	15.2	26.2	42.3	65.3	....	....	....	....	11.6	15.4	36.0	62	100	155
152	Kansas City, Mo.	963	33	50	1922	0.57	1.15	4.74	7.0	16.2	19.0	36.8	49.5	1.0	1.5	3.1	12.9	11.5	19.0	44.0	52	100	[illegible]
153	Oregon, Mo.	1 048	30	68	1922	....	....	....	7.0	15.0	21.4	35.6	49.7	0.9	....	....	....	13.6	19.7	42.1	60	100	140
154	Leavenworth, Kans.	913	....	87	1922	....	....	....	....	15.8	14.6	34.4	59.6	....	....	....	....	14.5	....	46.0	42	100	173
155	Manhattan, Kans.	1 031	29	65	1922	....	....	....	5.9	13.0	15.1	31.4	44.8	....	....	....	....	14.4	18.8	41.4	48	100	143
156	Dodge City, Kans.	2 509	48	49	1922	0.50	1.25	3.47	6.0	12.8	10.1	20.1	33.5	1.3	2.7	6.2	17.3	14.4	29.8	62.7	50	100	166
157	Wallace, Kans.	3 303	30	53	1922	....	....	....	6.1	12.6	7.4	16.2	34.0	....	....	....	....	19.4	37.6	77.8	46	100	210
158	North Platte, Nebr.	2 821	....	52	1925	....	....	....	....	8.5	11.2	18.4	32.7	1.2	....	....	....	14.8	....	46.2	61	100	178
159	Lincoln, Nebr.	1 189	30	51	1926	....	....	....	8.4	14.2	15.1	28.1	41.2	1.0	....	....	....	12.9	29.9	50.6	54	100	147
160	Omaha, Nebr.	1 103	57	57	1927	0.67	1.27	2.53	7.0	12.7	15.5	28.5	48.9	1.0	2.3	4.5	8.9	16.6	24.5	44.6	54	100	172
161	Des Moines, Iowa	861	42	43	1920	0.66	1.36	2.65	5.1	15.8	18.2	32.1	56.8	0.9	2.1	4.2	8.3	14.6	15.9	49.2	57	100	177
161-A	Cedar Rapids, Iowa	733	39	39	1920	....	....	....	7.8	15.4	18.7	31.1	44.9	....	....	....	....	10.8	25.1	49.5	60	100	144
162	Dubuque, Iowa	698	18	65	1920	0.80	1.52	2.70	5.4	14.2	19.3	33.4	55.1	0.9	2.4	4.6	8.1	13.2	16.2	42.5	58	100	165
163	Davenport, Iowa	606	55	55	1926	0.54	1.38	2.71	5.2	12.7	17.3	32.4	46.8	0.9	1.7	4.3	8.4	10.9	26.1	39.2	52	100	145
164	Keokuk, Iowa	614	13	50	1920	0.56	1.18	2.38	4.8	12.7	21.8	33.6	51.6	1.0	1.7	3.5	7.1	11.4	14.3	37.8	65	100	154
165	Springfield, Ill.	644	48	49	1927	0.66	1.41	2.75	5.9	15.2	22.8	36.4	58.2	0.9	1.8	3.9	7.6	10.1	16.2	41.8	63	100	160
166	Peoria, Ill.	609	....	72	1927	0.48	1.04	2.24	5.5	12.3	23.2	34.7	53.3	0.9	1.4	3.0	6.4	11.1	15.8	35.5	67	100	154
167	Chicago, Ill.	824	50	84	1927	0.61	1.24	2.13	6.2	11.8	22.4	33.1	47.1	0.8	1.8	3.7	6.4	9.5	18.7	34.1	68	100	142
168	Fort Wayne, Ind.	856	24	52	1928	0.81	1.27	2.27	4.9	10.6	26.7	37.1	47.0	....	2.2	3.4	6.1	10.9	13.1	28.6	72	100	127
169	Indianapolis, Ind.	822	52	57	1927	0.83	1.30	2.66	6.8	13.1	30.3	40.4	57.6	0.8	2.0	3.2	6.6	9.7	16.8	32.5	75	100	143
170	Terre Haute, Ind.	576	22	34	1928	1.15	1.38	2.57	5.2	11.4	29.3	38.8	48.3	0.8	3.0	3.6	6.7	9.4	13.5	29.6	76	100	126
171	Cincinnati, Ohio	628	51	93	1927	0.78	1.78	2.60	5.2	11.7	18.0	40.7	65.2	0.8	1.9	4.4	6.4	8.9	12.8	28.7	44	100	160
172	Portsmouth, Ohio	527	....	92	1921	....	....	....	3.5	11.1	24.3	41.2	57.6	0.9	....	....	....	8.2	8.5	27.0	59	100	140
173	Marietta, Ohio	627	28	104	1921	....	....	....	4.3	12.4	26.0	41.2	61.8	0.8	....	....	....	11.2	10.4	30.1	63	100	150
174	Columbus, Ohio	918	44	50	1927	0.38	1.00	2.80	3.8	9.6	26.5	36.4	51.3	0.7	1.0	2.7	7.7	9.9	10.4	26.4	73	100	141
175	Cleveland, Ohio	762	51	56	1926	0.44	1.27	1.80	5.0	9.8	24.5	34.1	53.5	0.6	1.3	3.7	5.6	9.6	14.6	28.7	72	100	157
176	Sandusky, Ohio	629	44	61	1927	0.56	1.34	2.22	4.2	10.6	23.5	33.1	46.3	0.7	1.7	4.0	6.7	8.9	12.7	32.0	71	100	140
177	Toledo, Ohio	769	51	60	1920	0.60	1.77	3.58	6.0	10.2	21.3	31.7	45.9	0.8	1.9	5.6	11.3	8.0	15.9	32.1	67	100	146
178	Detroit, Mich.	730	57	57	1927	....	....	....	4.7	8.8	21.1	32.0	47.7	0.7	....	....	....	10.4	14.7	27.5	66	100	149
179	Lansing, Mich.	863	....	65	1927	....	....	....	....	11.3	19.4	30.8	48.4	0.9	....	....	....	11.7	....	36.7	63	100	157
180	Alpena, Mich.	609	56	56	1927	....	....	....	3.9	13.2	17.4	30.7	45.6	0.7	....	....	....	9.8	12.7	43.0	57	100	149
181	Marquette, Mich.	734	....	55	1925	....	....	....	....	8.6	19.7	31.0	41.2	0.6	....	....	....	11.1	....	27.7	64	100	133
182	Milwaukee, Wis.	681	57	57	1927	....	....	....	5.8	10.0	18.7	30.8	50.4	0.8	....	....	....	11.3	18.8	33.5	61	100	164
183	Madison, Wis.	974	61	61	1920	....	....	....	5.2	10.7	13.5	31.2	52.9	0.9	....	....	....	10.3	16.7	34.3	43	100	169
184	La Crosse, Wis.	714	....	48	1920	0.31	0.87	1.80	7.2	12.1	16.8	31.1	44.7	0.9	1.0	2.8	5.8	7.8	23.2	38.9	54	100	144
185	St. Paul, Minn.	940	56	90	1926	0.40	1.00	2.20	5.7	11.7	10.2	27.2	49.7	0.8	1.5	3.7	8.1	15.2	20.9	48.9	37	100	183
186	Duluth, Minn.	1 133	50	57	1927	0.41	1.06	2.26	5.2	13.3	18.1	28.5	45.3	0.8	1.4	3.7	7.9	12.2	18.6	46.7	64	100	159
187	Moorhead, Minn.	935	46	46	1926	0.53	1.12	2.21	5.2	9.6	9.9	23.1	34.0	0.9	2.3	4.4	9.6	12.4	22.5	41.6	43	100	147
188	Bismarck, N. Dak.	1 674	....	51	1925	....	....	....	....	9.9	11.0	17.0	30.9	1.1	....	....	....	20.9	....	53.2	65	100	182
189	Williston, N. Dak.	1 875	20	27	1920	....	....	....	4.2	8.8	8.6	14.4	22.0	1.1	....	....	....	22.8	29.1	61.0	60	100	153
190	Huron, S. Dak.	1 306	39	40	1920	....	....	....	3.9	11.6	10.2	20.9	30.1	1.0	....	....	....	18.4	18.6	56.4	49	100	144
191	Pierre, S. Dak.	1 572	29	30	1920	....	....	....	3.7	6.1	7.8	17.0	23.6	1.2	....	....	....	16.5	21.8	35.9	46	100	139
192	Rapid City, S. Dak.	3 251	33	33	1920	....	....	....	4.8	9.7	9.6	17.7	27.1	1.0	....	....	....	13.6	27.1	54.8	54	100	153
193	Sheridan, Wyo.	3 790	15	28	1920	0.49	0.85	1.32	2.1	6.8	8.0	14.4	22.5	0.9	2.4	5.9	9.2	18.6	14.6	45.8	56	100	156
194	Cheyenne, Wyo.	6 088	50	50	1920	0.30	0.88	1.18	4.7	7.7	5.0	14.0	22.7	1.1	2.1	6.3	8.4	12.9	33.6	55.0	36	100	162
195	Yellowstone Park, Wyo.	6 200	32	40	1927	0.23	0.33	0.63	2.3	8.9	7.9	18.0	25.6	0.8	1.3	1.8	3.5	8.2	12.8	40.4	44	100	150
196	Miles City, Mont.	2 371	31	43	1920	....	....	....	3.4	5.9	9.1	13.5	22.7	1.2	....	....	....	11.7	25.2	49.7	67	100	168
197	Havre, Mont.	2 505	19	41	1920	0.43	0.77	1.60	3.7	9.7	6.8	13.5	25.7	1.1	3.2	5.7	11.9	13.6	27.4	71.3	50	100	191
198	Kalispell, Mont.	2 973	24	26	1921	0.48	0.90	1.02	1.8	5.2	10.7	15.2	19.7	0.8	2.8	5.9	6.8	10.1	11.8	34.2	70	100	130
199	Helena, Mont.	4 110	42	47	1926	0.24	0.49	1.04	3.7	6.2	6.7	13.4	20.0	1.0	1.8	3.7	7.8	15.9	27.6	46.2	50	100	149
200	Pocatello, Idaho	4 477	....	....	1920	....	....	....	1.6	4.3	9.8	13.9	23.4	1.1	....	....	....	11.3	11.5	30.9	67	100	161
201	Boise, Idaho	2 739	46	64	1927	....	....	....	2.7	7.7	6.0	13.5	25.8	1.1	....	....	....	11.6	30.0	57.0	44	100	191
202	Lewiston, Idaho	757	....	41	1920	....	....	....	1.9	6.3	9.1	13.4	21.7	0.9	....	....	....	10.5	14.2	47.0	68	100	162
203	Spokane, Wash.	1 943	19	46	1926	0.29	0.38	0.77	2.2	5.8	9.9	16.2	26.0	0.9	1.8	2.3	4.7	12.9	18.5	35.6	61	100	160
204	Seattle, Wash.	248	34	37	1927	0.29	0.52	0.80	3.5	11.8	21.7	32.9	45.8	0.7	0.9	1.6	2.4	16.1	10.6	35.9	66	100	139
205	Forks (N. W.), Wash.	480	....	15	1921	....	....	....	....	31.3	108.4	122.3	153.6	0.6	....	....	....	14.8	....	25.6	89	100	126
206	North Head, Wash.	211	19	44	1926	0.27	0.37	0.88	4.3	17.3	39.9	55.5	78.6	0.5	0.5	0.7	1.6	16.5	7.7	31.2	72	100	142
207	Astoria, Ore.	24	....	66	1922	....	....	....	....	24.7	49.4	77.0	101.4	0.5	....	....	....	15.7	....	32.1	64	100	132
208	Portland, Ore.	57	57	57	1927	0.40	0.85	1.31	7.7	20.1	30.8	42.8	67.2	0.6	0.9	1.9	3.1	15.4	18.0	47.0	72	100	157
209	Cascade Locks, Ore.	100	....	44	1922	....	....	....	....	35.1	60.1	78.4	100.6	0.6	....	....	....	16.7	....	44.8	64	100	128
210	The Dalles, Ore.	96	....	65	1922	....	....	....	....	10.8	7.5	16.3	48.6	....	....	....	....	16.9	....	66.2	46	100	[illegible]
211	Baker, Ore.	3 471	31	31	1920	....	....	....	1.8	4.0	7.7	12.2	18.2	1.0	....	....	....	10.7	14.6	32.8	63	100	149
212	Roseburg, Ore.	510	45	45	1922	0.56	1.10	1.14	4.1	12.8	21.1	33.3	44.3	0.8	1.7	3.3	3.4	15.7	12.3	38.5	63	100	133
213	Monumental, N.W., Calif.	2 700+	....	6	1910	....	....	....	....	43.8	69.3	109.1	152.5	....	....	....	....	19.4	....	40.0	68	100	140
214	Crescent City, Calif.	125	....	34	1922	....	....	....	....	31.9	47.5	74.8	107.6	0.9	....	....	....	18.5	....	42.6	63	100	144
215	Fort Gaston, Calif.	397	....	30	1892	....	....	....	....	24.5	34.8	54.7	71.0	....	....	....	....	13.6	....	63.0	64	100	130
216	Eureka, N. W., Calif.	64	37	37	1922	....	....	....	5.1	19.5	22.4	41.8	64.5	0.8	....	....	....	16.3	12.2	46.6	54	100	154
217	Weaverville, Calif.	2 162	....	34	1922	....	....	....	....	19.8	21.6	37.2	54.8	....	....	....	....	19.4	....	53.2	58	100	147
218	Delta, Calif.	1 138	....	34	1916	....	....	....	....	53.3	28.1	65.0	114.8	....	....	....	....	19.6	....	82.2	43	100	177
219	Red Bluff, Calif.	307	47	48	1925	0.26	0.57	1.72	6.1	20.7	12.9	24.6	49.0	1.4	1.1	3.3	7.0	19.2	24.7	84.0	52	100	199
220	Marysville, Calif.	67	....	52	1922	....	....	....	....	14.4	10.8	19.6	36.7	....	....	....	....	20.2	....	73.5	55	100	187
221	Helen Mine, Calif.	2 750	....	22	1922	....	....	....	....	71.5	50.8	86.3	137.6	....	....	....	....	23.9	....	83.0	59	100	160
222	Santa Rosa, Calif.	181	....	35	1922	....	....	....	....	18.4	13.6	29.9	46.5	1.4	....	....	....	22.4	....	65.0	46	100	156
223	Fort Ross, Calif.	100	....	49	1922	....	....	....	....	37.4	21.7	51.9	88.2	....	....	....	....	21.9	....	72.0	42	100	170
224	Point Reyes, Calif.	490	32	43	1922	0.20	0.69	1.40	3.1	11.4	7.3	20.6	32.4	1.3	1.0	3.4	6.8	16.6	15.0	55.2	36	100	157
225	Mt. Tamalpais, Calif.	2 375	22	26	1922	....	....	....	3.0	15.6	13.5	27.5	42.2	1.2	....	....	....	21.2	10.9	56.8	49	100	153
226	San Francisco, Calif.	206	57	79	1927	....	....	....	4.7	24.4	9.0	22.5	38.8	1.4	....	....	....	21.3	20.9	108.0	40	100	172
227	San Jose, Calif.	95	17	49	1922	....	....	....	4.6	12.4	6.6	15.1	25.5	1.6	....	....	....	20.2	30.4	82.0	44	100	169
228	Boulder Creek, Calif.	470	....	29	1916	....	....	....	....	39.4	21.2	55.6	95.8	....	....	....	....	25.4	....	71.0	38	100	172
229	Monterey, Calif.	18	....	51	1915	....	....	....	....	11.7	7.2	16.7	29.0	....	....	....	....	20.8	....	70.0	43	100	174
230	San Luis Obispo, Calif.	201	28	56	1925	....	....	....	6.0	18.2	6.9	21.1	50.0	1.9	1.2	1.9	5.8	23.8	28.4	96.0	33	100	237
231	Santa Barbara, Calif.	130	....	56	1922	....	....	....	....	17.2	7.2	18.7	43.2	2.3	....	....	....	23.8	....	92.0	39	100	231
232	Los Angeles, Calif.	361	47	51	1927	....	....	....	5.1	15.8	4.8	15.2	40.3	2.6	....	....	....	17.4	33.5	[illegible]	32	100	265
233	Lowe Observatory, Calif.	3 420	....	24	1919	....	....	....	....	19.9	8.3	26.7	58.5	....	....	....	....	22.7	....	74.5	31	100	219
234	Mount Wilson, Calif.	5 850	....	16	1919	....	....	....	....	24.5	8.2	33.8	52.9	....	....	....	....	25.3	....	72.4	24	100	156
235	Riverside, Calif.	851	....	40	1919	....	....	....	....	9.7	3.9	10.7	25.3	2.6	....	....	....	21.7	....	90.6	36	100	236
236	San Diego, Calif.	87	56	79	1927	....	....	....	3.2	9.3	4.3	9.8	27.6	2.3	....	....	....	19.2	32.6	95.0	44	100	282
237	Campo, Calif.	2 543	....	21	1919	....	....	8.6	....	20.4	11.3	19.8	35.3	....	....	....	43.5	18.7	....	103.0	57	100	178
238	Cuyamaca, Calif.	4 677	....	33	1919	....	....	....	....	36.5	19.2	38.8	66.1	1.7	....	....	....	19.8	....	96.9	49	100	170
239	Sterling, Calif.	258	....	42	1919	....	....	....	....	5.4	Tr.	2.2	7.4	....	....	....	....	19.0	....	245.0	0	100	[illegible]
240	Indio, Calif.	—20	....	43	1919	....	....	....	....	6.0	0.1	2.8	7.1	8.3	....	....	....	29.3	....	214.4	4	100	[illegible]

(Continued.)

Station No.	January.			February.			March.			April.			May.			June.			July.			August.			September.			October.			November.			December.		
	Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.	
		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
119	4.56	4.2	11.1	4.17	6.7	12.8	4.64	8.7	15.9	4.82	14.0	14.9	4.42	9.1	18.7	5.47	5.8	24.6	6.54	7.5	15.5	5.73	8.9	22.7	4.61	9.2	13.9	3.47	7.8	13.4	3.08	4.4	8.3	4.67	4.6	14.4
120	3.97	7.5	12.1	3.46	3.5	8.7	4.32	4.5	11.7	4.75	4.6	11.0	4.24	8.3	14.5	3.46	8.0	8.8	3.48	5.6	13.2	2.60	6.5	10.9	3.04	7.0	16.5	2.88	4.2	11.7	3.64	5.3	9.5	4.45	7.2	15.5
121	3.58	2.7	8.5	2.35	1.4	10.7	3.37	4.4	9.5	4.26	6.1	13.0	4.27	4.4	12.9	3.38	4.3	10.2	2.89	2.1	11.9	3.08	5.2	13.7	2.73	2.5	7.6	2.73	3.7	10.9	2.76	4.4	10.1	2.47	3.8	7.2
122	1.51	3.1	4.7	1.52	2.8	4.6	2.18	4.3	7.6	4.12	8.8	17.6	4.36	7.0	10.7	3.08	3.9	8.7	2.57	3.2	8.3	2.72	5.4	10.3	2.46	7.4	9.1	2.09	4.0	9.4	2.57	4.7	7.9	1.84	2.8	5.4
123	3.29	4.6	11.1	2.98	3.7	7.0	3.36	4.7	9.5	4.27	4.6	11.2	4.52	5.7	17.2	3.42	4.7	9.8	2.75	6.3	10.0	2.44	5.2	9.8	2.93	6.3	10.6	3.35	5.3	10.0	3.52	5.2	10.1	3.91	5.4	11.0
124	1.94	1.9	7.7	1.77	3.1	7.1	1.95	2.5	6.1	3.96	9.8	16.2	4.16	4.7	9.5	2.66	4.3	8.8	2.24	6.5	14.4	2.07	4.8	9.1	3.46	23.1	25.9	2.97	7.5	9.8	2.41	3.2	8.2	2.42	6.0	11.7
125	1.97	...	8.0	2.33	...	9.4	2.31	...	7.2	3.51	...	19.8	4.28	...	14.1	2.48	...	8.4	2.38	...	12.8	2.40	...	10.9	3.45	...	12.8	3.00	...	12.5	2.50	...	8.0	2.37	...	18.1
126	3.89	4.5	13.1	3.04	3.1	9.0	3.48	7.4	10.7	2.78	4.7	10.6	4.93	5.4	19.2	4.87	7.2	15.8	3.88	4.4	14.8	4.13	7.5	15.4	4.15	6.1	13.4	3.70	6.3	9.8	3.57	5.2	10.8	4.24	7.5	14.4
127	3.62	3.4	10.4	3.10	3.8	9.3	2.90	4.0	8.1	3.13	9.2	11.0	3.23	6.1	20.3	4.75	10.3	15.5	3.98	14.2	18.7	5.01	9.0	19.1	6.41	10.0	26.0	4.18	14.1	17.8	4.02	5.6	9.2	3.73	4.5	10.8
128	1.86	2.2	8.6	1.60	3.4	7.9	1.63	2.2	5.3	2.90	6.8	11.6	3.07	8.0	7.9	2.53	3.6	9.4	2.84	3.4	7.9	2.58	4.5	9.1	3.30	4.7	11.7	2.44	7.1	8.9	2.18	3.0	9.2	1.77	2.6	7.3
129	1.47	3.0	5.9	1.48	5.8	6.8	1.56	4.7	7.7	1.88	4.2	6.3	3.08	6.6	8.7	2.04	8.3	9.4	1.39	4.3	6.9	1.77	6.1	7.6	4.08	6.2	18.8	2.25	4.2	7.3	1.97	4.6	8.6	1.50	2.5	6.1
130	1.35	...	4.6	1.26	...	8.0	1.22	...	6.5	1.30	...	6.0	2.23	...	9.0	2.46	...	12.8	1.88	...	6.8	2.54	...	16.6	5.32	...	30.6	3.23	...	16.3	2.07	...	5.3	1.40	...	6.3
131	0.72	...	4.6	0.72	...	4.5	1.06	...	5.6	1.57	...	8.1	2.94	...	8.2	2.57	...	13.5	1.85	...	7.5	2.58	...	9.1	2.92	...	8.2	1.87	...	7.7	1.18	...	3.6	0.82	...	6.6
132	0.46	1.3	1.6	0.41	1.1	1.9	0.36	1.3	2.1	0.27	1.0	1.4	0.31	2.1	2.3	0.59	1.8	2.6	1.99	6.5	9.2	1.69	2.4	4.4	1.38	2.1	3.7	0.82	1.8	5.1	0.50	2.0	2.5	0.58	1.6	3.9
133	0.91	2.0	3.1	0.93	3.2	4.1	1.24	2.5	4.0	2.78	4.6	9.8	3.82	6.5	8.4	2.77	4.0	8.4	1.89	3.8	7.8	2.33	4.1	15.7	2.54	4.6	9.6	2.55	4.6	7.6	1.34	2.1	5.8	1.30	4.6	6.7
134	0.64	1.9	2.4	0.72	1.9	2.9	0.77	3.1	4.1	2.01	2.7	6.1	2.98	3.9	9.1	3.01	3.9	9.8	2.87	3.0	7.3	3.21	4.0	6.8	2.37	3.5	6.4	1.79	2.2	7.2	0.92	3.0	5.1	0.83	2.6	3.0
135	1.17	...	4.4	1.10	...	3.1	2.10	...	4.9	2.14	...	7.5	5.40	...	12.0	3.49	...	12.8	2.87	...	7.3	2.88	...	8.3	2.62	...	7.3	2.57	...	8.4	2.07	...	7.8	1.86	...	5.8
136	2.27	...	12.1	2.02	...	10.4	2.76	...	8.4	4.41	...	12.6	4.72	...	12.9	3.63	...	9.2	2.63	...	11.9	3.56	...	13.1	3.02	...	8.0	3.34	...	9.3	2.76	...	8.1	3.10	...	7.7
137	3.58	3.5	11.2	2.16	2.1	3.1	3.78	4.4	9.5	4.46	3.0	8.2	4.70	3.2	10.1	4.08	3.9	9.2	3.54	5.0	7.5	4.04	4.8	9.7	3.20	4.6	5.9	3.10	3.6	10.7	2.96	3.1	6.1	2.19	1.9	3.0
138	3.46	3.6	7.3	2.54	3.0	10.7	3.17	3.8	10.5	3.82	5.0	8.2	4.63	5.1	10.6	3.71	4.2	7.7	3.35	3.9	15.0	3.49	5.1	10.9	2.84	3.1	7.2	2.83	4.8	7.4	[illegible]	[illegible]	7.4	2.58	3.5	6.1
139	4.66	5.8	11.8	3.29	5.2	12.7	4.64	3.2	10.4	5.13	3.6	14.8	4.70	5.3	15.9	3.74	4.4	9.3	3.47	5.2	9.2	3.75	4.1	11.1	3.16	4.7	10.2	2.72	6.3	15.3	4.13	5.9	10.2	4.13	5.9	16.9
140	4.94	4.0	12.9	4.32	3.2	9.9	5.47	9.3	18.0	5.06	5.0	12.9	4.20	4.3	13.3	3.85	9.7	18.2	3.19	3.7	7.5	3.40	4.5	10.6	2.91	4.7	10.8	2.78	4.3	10.1	4.21	6.5	14.5	4.57	5.4	11.9
141	4.70	3.5	14.6	4.14	5.2	12.4	5.11	4.5	10.3	4.38	5.0	11.8	3.75	2.8	8.7	4.08	4.4	10.3	3.99	5.1	9.4	3.63	5.6	9.6	3.39	4.9	10.9	2.47	2.5	8.3	3.84	6.0	8.9	4.67	5.1	12.8
142	5.28	3.6	14.7	4.93	3.4	10.3	5.83	7.6	14.0	4.78	6.6	15.3	3.79	3.7	8.3	4.17	2.4	9.2	4.25	4.9	12.5	4.05	3.3	12.4	3.28	4.8	7.9	2.92	2.1	6.8	3.16	2.0	8.8	4.86	2.6	8.8
143	4.78	3.9	17.0	4.57	4.4	12.5	5.18	5.6	13.3	4.48	4.2	17.3	3.77	3.0	7.3	4.40	3.0	9.7	4.33	6.2	13.2	4.07	3.0	11.3	2.84	5.7	9.0	2.61	3.1	7.9	3.22	3.4	7.2	4.31	3.8	12.8
144	4.17	3.5	10.3	3.86	3.0	11.1	4.33	2.6	9.9	3.77	3.3	7.4	3.89	2.6	11.0	4.16	4.4	10.4	3.85	4.0	11.1	3.56	3.0	8.2	3.21	3.4	8.6	2.46	2.3	7.9	3.37	3.9	8.5	3.88	2.6	9.0
145	4.06	3.6	12.1	3.52	6.0	9.8	4.40	5.1	9.9	3.96	4.1	9.1	3.69	3.9	9.6	3.96	3.0	7.0	3.91	5.6	16.6	3.88	3.8	10.5	2.74	3.1	5.9	2.71	5.1	8.0	3.59	3.2	8.3	3.78	4.4	8.4
146	3.72	2.7	10.3	3.20	3.1	14.6	4.20	4.0	11.0	3.91	3.9	10.1	3.81	3.8	8.4	4.04	4.9	11.4	3.42	4.7	10.3	3.36	3.8	9.8	3.31	4.4	7.3	2.78	6.9	11.2	3.76	3.8	10.0	3.51	3.0	7.0
147	3.99	5.2	15.0	3.17	3.2	10.1	3.92	4.3	9.8	3.81	3.6	9.6	3.72	4.2	10.8	3.92	5.1	10.1	3.22	3.4	9.9	2.96	5.4	12.0	2.82	4.5	7.3	2.75	5.7	11.6	3.71	4.5	11.3	3.38	3.3	9.0
148	2.36	3.6	8.5	2.52	4.4	8.9	3.54	3.9	8.6	3.76	4.0	10.8	4.52	3.4	11.8	4.04	4.6	17.1	3.55	5.1	9.5	3.47	7.0	11.4	3.25	3.7	10.5	2.86	3.4	8.7	2.86	3.6	8.6	2.51	2.0	10.9
149	1.72	3.1	6.4	1.72	1.6	3.9	2.54	3.6	6.4	3.16	3.0	6.4	4.46	4.1	8.8	3.81	2.9	9.7	3.29	5.4	9.4	3.19	4.6	9.1	3.77	3.8	11.6	2.27	3.7	10.7	1.95	2.2	4.7	1.42	2.7	6.0
150	3.06	3.9	6.9	1.89	2.8	6.5	3.03	3.9	8.5	3.87	2.8	11.3	4.52	3.2	10.6	4.44	2.9	9.2	3.31	3.3	7.7	3.65	3.7	7.8	4.50	6.6	11.5	2.39	3.3	7.7	2.06	3.6	6.4	1.70	3.9	7.8
151	2.85	4.6	9.8	2.20	2.2	7.3	3.46	3.8	9.0	3.96	2.4	9.6	5.27	4.2	11.6	4.80	3.4	15.2	4.22	5.5	13.1	4.17	3.2	10.8	3.48	4.8	8.5	3.08	4.1	11.9	2.82	3.8	6.3	2.23	5.0	11.0
152	1.26	2.2	5.5	1.72	2.1	6.8	2.50	2.7	8.7	3.11	3.2	7.2	4.73	4.2	11.0	4.30	4.4	10.9	4.18	5.6	9.6	4.06	5.9	12.3	4.24	7.0	10.2	2.95	4.9	8.5	1.91	2.4	6.0	1.35	1.6	5.1
153	1.32	1.0	6.2	1.62	1.8	4.9	2.03	3.1	5.3	3.29	2.6	7.8	4.90	3.4	15.0	4.72	6.9	14.9	4.22	7.0	13.2	3.97	4.6	8.5	3.74	5.3	13.0	2.86	2.5	6.5	1.80	3.0	7.8	1.44	1.5	5.8
154	1.08	...	4.2	1.37	...	5.6	1.94	...	5.8	3.00	...	7.6	4.40	...	12.4	4.90	...	10.8	3.90	...	10.6	2.97	...	10.2	3.80	...	9.2	2.41	...	8.7	2.24	...	7.8	1.10	...	5.3
155	0.75	1.2	3.6	1.23	1.9	5.8	1.51	2.4	6.5	3.67	3.7	9.1	4.54	5.0	13.0	4.50	4.7	10.3	4.60	5.6	12.7	3.63	5.9	10.7	3.38	4.5	8.1	2.24	2.5	9.1	1.46	2.2	8.0	0.90	1.4	3.5
156	0.42	1.8	1.8	0.78	2.1	2.9	0.84	1.9	3.8	1.83	2.9	5.5	2.90	5.3	12.6	3.32	6.0	11.2	3.00	3.5	7.3	2.61	2.9	6.2	1.76	5.9	9.3	1.30	3.2	3.9	0.71	3.1	5.6	0.65	1.8	4.6
157	0.31	0.8	1.2	0.44	0.8	1.6	0.48	0.8	1.7	1.74	2.2	6.3	2.46	5.1	9.5	2.46	3.0	8.9	3.14	4.1	12.6	2.10	3.4	6.2	1.30	4.4	6.1	0.93	1.8	3.3	0.44	1.0	2.5	0.50	0.7	2.3
158	0.41	...	2.2	0.57	...	1.6	0.83	...	2.2	2.08	...	7.1	2.88	...	8.0	3.06	...	7.6	2.73	...	8.5	2.28	...	6.6	1.38	...	5.1	1.08	...	4.1	0.47	...	2.8	0.50	...	3.9
159	0.66	0.7	1.8	0.98	1.9	2.9	1.22	2.0	3.7	2.49	2.8	6.1	4.12	3.0	11.3	4.83	4.2	13.1	3.38	5.1	11.3	3.63	3.4	14.1	3.02	4.0	8.1	1.92	2.2	6.0	1.05	2.8	7.1	0.82	1.5	4.0
160	0.66	0.6	2.2	0.86	1.6	3.1	1.37	1.6	4.0	3.64	2.8	6.4	3.85	3.0	11.3	4.74	6.0	12.7	3.80	4.8	10.3	3.12	7.6	12.5	3.28	4.8	9.3	2.20	3.7	5.8	1.08	2.8	6.3	0.91	1.3	2.7
161	1.15	...	3.6	1.18	...	3.2	1.64	...	3.9	3.02	...	7.4	4.72	...	10.6	4.06	...	15.8	3.65	...	10.2	3.35	...	8.0	3.42	...	14.8	2.59	...	6.4	1.49	...	6.8	1.27	...	3.2
161-A	1.35	...	3.2	1.40	...	5.0	1.88	...	4.5	2.81	...	7.8	4.45	...	10.1	3.01	4.6	8.8	3.67	4.5	8.1	2.88	...	9.9	3.86	7.8	16.4	2.31	...	4.9	1.35	...	4.1	1.31	...	3.0
162	1.59	2.6	3.6	1.52	1.3	6.4	2.16	2.5	5.0	2.66	4.6	7.6	4.05	3.2	11.0	4.61	3.5	14.2	3.90	4.8	10.5	3.31	5.2	8.7	3.78	5.4	10.3	3.62	3.4	6.7	1.82	2.0	5.4	1.82	1.7	4.1
163	1.51	1.8	4.6	1.51	2.9	4.9	2.29	3.0	6.0	2.77	3.0	6.0	3.95	2.8	9.2	4.17	5.1	10.8	3.49	5.2	9.4	3.52	4.6	12.7	3.61	3.6	8.8	2.29	4.1	7.2	1.80	2.5	4.7	1.51	1.9	3.8
164	1.69	2.8	5.2	1.64	3.6	6.1	2.29	3.6	6.0	3.10	2.6	6.1	4.18	3.1	11.5	4.29	3.6	10.0	3.88	4.3	12.7	2.92	3.7	6.9	3.86	4.8	11.1	2.30	2.2	9.0	1.92	1.8	4.0	1.59	2.8	6.0
165	2.12	2.1	6.2	2.34	3.6	7.9	3.17	4.7	9.6	3.46	2.7	10.2	4.38	5.8	11.8	4.06	5.9	12.7	2.86	3.2	8.5	2.14	4.8	7.1	3.67	5.5	15.2	2.50	3.2	10.0	2.51	3.2	6.9	2.16	3.7	6.1
166	1.76	2.8	5.8	1.93	2.8	5.6	2.75	2.8	5.8	3.23	2.7	7.9	3.82	6.6	11.6	3.72	4.7	11.2	3.78	4.1	8.9	3.22	3.7	9.0	3.82	3.6	12.3	2.31	2.0	5.7	3.32	3.0	5.5	2.00	2.6	7.1
167	1.97	1.9	6.6	2.05	1.8	6.0	2.62	3.3	5.6	2.83	2.7	7.6	3.62	2.8	7.8	3.44	3.4	10.6	3.38	4.1	8.6	3.15	6.2	11.3	3.15	3.4	8.3	2.57	2.5	7.8	2.41	3.4	6.1	2.04	2.7	6.4
168	2.48	2.0	6.0	2.34	2.0	5.4	3.42	3.0	7.9	3.07	2.8	7.7	3.72	3.4	7.4	3.88	4.4	10.2	4.06	2.8	10.6	3.22	4.8	6.1	3.05	4.2	9.9	2.84	2.6	7.6	2.09	1.8	7.6	2.72	1.7	9.0
169	2.98	4.4	10.1	2.72	3.0	7.2	4.02	5.4	10.9	3.66	3.2	8.6	3.85	3.3	8.8	3.80	2.9	12.2	3.90	4.2	12.1	3.33	3.1	7.8	3.24	3.8	9.3	2.76	3.4	5.9	2.30	4.3	7.8	2.96	3.5	6.2
170	2.78	3.2	7.7	2.20	2.1	6.1	4.07	3.8	11.0	3.61	3.8	10.6	3.74	3.2	9.8	3.90	3.7	7.9	3.16	3.7	7.9	3.35	2.7	7.0	3.62	5.2	11.4	2.67	2.4	8.8	2.86	3.1	7.1	2.87	3.4	7.0
171	3.35	3.6	9.8	3.01	3.0	8.9	3.77	5.2	9.9	3.37	2.5	7.7	3.85	2.2	9.5	4.08	2.8	11.5	3.90	4.1	10.0	3.62	2.8	11.7	2.86	2.9	10.9	2.71	4.3	9.6	3.07	2.8	6.8	3.82	3.1	10.8
172	3.04	2.7	8.2	3.17	3.4	10.2	3.80	2.9	8.4	3.29	2.8	8.8	3.58	2.8	10.6	4.30	3.1	10.7	4.06	2.7	9.4	3.59	3.5	7.6	2.73	2.0	9.3	2.76	3.4	9.6	2.96	3.2	7.8	3.39	3.8	11.1
173	3.17	1.6	8.1	3.12	1.6	10.2	3.48	2.8	7.6	3.41	3.3	7.6	3.85	3.1	12.4	4.31	2.1	8.6	4.01	4.8	12.4	3.84	3.8	11.8	3.07	2.3	10.3	2.80	2.1	7.7	2.92	2.6	8.6	3.41	1.8	10.8
174	3.07	2.0	6.6	2.70	2.5	7.6	3.52	3.8	8.1	2.90	3.3	7.1	3.60	2.4	9.6	3.30	3.6	8.5	3.55	3.6	9.6	3.36	3.7	7.2	2.57	2.4	7.1	2.46	3.2	8.6	2.77	2.8	7.8	2.72	2.6	6.1
175	2.62	1.9	8.7	2.42	3.6	7.7	2.75	3.2	8.3	2.46	2.0	8.4	3.09	2.2	7.4	3.27	3.1	9.8	3.50	3.8	8.1	2.96	4.1	8.8	3.36	5.0	9.3	2.81	2.6	6.7	2.81	2.2	8.4	2.40	1.8	8.5
176	2.36	1.7	8.1	2.18	3.0	8.5	2.71	3.0	8.7	2.55	2.1	6.2	3.16	2.1	7.1	3.68	2.5	10.1	3.44	3.6	7.9	3.16	4.2	8.0	2.96	2.4	10.6	2.44	2.8	6.2	3.16	2.2	6.4	2.28	1.7	6.3
177	2.23	1.6	6.6	2.01	2.8	6.8	2.42	2.7	8.7	2.52	2.6	5.8	3.46	3.6	7.6	3.41	2.3	8.2	3.10	2.3	6.6	2.87	4.6	7.6	2.52	6.0	10.2	2.43	3.1	8.5	2.45	2.7	6.8	2.30	2.0	5.8
178	2.07	1.8	6.0	2.17	2.4	6.6	2.46	2.1	5.6	2.44	2.4	6.2	3.37	2.6	7.8	3.68	3.4	8.3	3.31	4.7	8.8	2.82	4.8	8.8	2.80	3.3	6.5	2.38	2.6	6.8	2.40	2.6	5.8	3.33	3.0	4.8
179	1.78	...	4.2	1.81	...	5.7	2.34	...	5.6	2.53	...	6.4	3.25	...	6.6	3.60	...	11.3	2.12	...	11.3	2.78	...	6.7	2.75	...	6.7	2.41	...	5.8	2.28	...	5.1	1.87	...	6.9
180	1.90	1.8	5.2	1.86	1.7	5.0	1.99	2.8	5.6	2.17	1.7	4.5	3.09	2.6	7.4	3.22	2.5	6.5	1.79	2.6	6.6	2.96	8.8	8.0	3.09	8.7	9.1	3.00	3.6	13.2	2.59	2.8	6.4	2.08	1.6	5.6
181	2.17	...	4.8	1.76	...	5.2	2.08	...	4.5	2.28	...	6.8	3.04	...	9.1	3.42	...	6.8	3.08	...	6.8	3.72	...	7.7	3.15	...	8.3	2.92	...	6.1	2.98	...	6.6	2.42	...	8.0
182	1.90	2.8	6.4	1.76	1.9	5.4	2.50	2.2	5.5	2.75	3.1	7.1	3.45	2.8	6.5	3.47	5.8	10.0	2.88	3.0	7.1	2.77	3.6	8.1	3.53	3.7	8.7	2.41	2.3	7.1	1.88	2.8	4.9	1.77	1.6	4.6
183	1.67	1.8	3.6	1.49	1.8	5.4	2.01	1.9	4.7	2.54	2.7	7.2	3.78	2.2	8.4	3.58	3.8	9.8	3.71	4.3	9.8	3.12	5.1	7.6	3.36	5.2	10.7	2.86	2.5	9.1	1.78	2.6	6.0	1.56	2.6	5.7
184	1.16	1.2	3.4	1.06	3.0	3.2	1.64	2.1	4.1	2.41	1.9	6.0	3.79	3.1	8.4	4.08	4.9	11.6	3.90	4.7	11.0	3.54	4.2	8.8	3.87	5.7	10.9	2.42	7.2	12.1	1.61	2.1	6.8	1.21	2.1	3.4
185	0.82	1.0	4.3	0.84	1.1	3.7	1.41	2.6	4.1	2.80	2.9	5.6	3.33	3.6	10.4	4.14	3.1	11.7	3.61	5.7	11.1	3.34	3.4	9.6	3.21	5.2	10.6	1.99	3.6	7.5	1.36	2.8	5.7	0.97	1.5	2.9
186	0.95	1.0	2.4	1.02	1.1	4.2	1.55	3.0	5.0	2.02	2.2	6.3	3.30	3.4	8.0	4.15	3.6	10.9	3.32	5.3	10.8	3.22	2.8	7.9	3.48	3.0	13.3	2.37	2.3	11.6	1.50	1.8	3.7	1.16	1.8	3.3
187	0.68	1.4	1.6	0.70	1.2	2.2	0.94	1.0	2.8	2.12	2.8	6.5	2.77	3.2	7.2	3.89	4.4	9.1	3.80	5.2	8.0	2.97	2.6	6.8	2.27	4.3	6.3	1.68	2.6	4.7	0.92	2.1	3.1	0.68	1.2	1.7
188	0.50	...	1.6	0.48	...	1.5	1.00	...	2.3	1.60	...	4.2	2.38	...	5.7	3.05	...	9.9	2.19	...	4.8	1.92	...	6.5	1.27	...	5.6	0.97	...	2.9	0.62	...	2.1	0.57	...	1.7
189	0.54	1.4	2.0	0.42	0.6	1.5	0.83	1.2	2.2	1.10	1.6	2.8	2.11	4.2	6.8	3.21	3.5	8.8	1.69	3.8	3.4	1.67	3.4	5.0	1.01	1.6	2.2	0.69	1.1	2.1	0.63	1.7	2.1	0.46	0.7	1.5
190	0.56	1.6	2.9	0.56	0.9	1.4	0.91	1.2	1.7	2.38	2.3	6.3	3.06	3.6	6.4	3.85	3.9	11.6	3.06	3.0	6.4	2.47	2.6	6.7	1.58	2.7	6.7	1.34	2.8	3.4	0.59	1.4	1.8	0.56	0.8	2.1
191	0.67	0.9	1.7	0.54	1.0	1.8	1.04	1.5	2.5	1.90	2.1	3.9	2.51	2.8	5.8	2.62	3.2	6.1	2.80	2.9	6.1	2.12	3.7	5.1	1.15	1.8	2.8	0.38	2.7	3.2	0.42	0.8	1.9	0.51	1.2	1.4
192	0.46	0.6	1.2	0.50	0.9	1.6	1.00	2.2	1.8	1.97	2.4	4.3	3.45	4.2	8.1	3.34	3.7	7.6	2.41	3.1	9.7	1.62	2.2	5.7	1.24	4.8	5.1	0.90	1.3	3.3	0.38	0.8	1.4	0.42	0.8	1.0
193	0.90	1.1	2.0	0.74	0.8	1.9	1.22	1.7	3.3	1.67	1.2	4.2	2.88	2.1	6.6	1.90	1.6	4.7	1.04	1.4	4.4	0.72	0.5	3.6	1.34	1.5	3.8	1.02	1.2	2.9	0.89	0.5	1.8	0.60	0.5	1.5
194	0.85	0.9	2.0	0.57	1.0	1.8	0.94	1.4	3.2	1.81	2.1	7.7	2.39	2.7	6.7	1.47	1.4	4.0	2.03	4.7	6.3	1.48	1.7	4.0	1.19	2.9	3.9	0.84	1.3	2.6	0.44	1.0	1.6	0.45	0.8	2.0
195	1.08	1.6	7.7	1.42	1.2	6.6	1.78	1.1	4.9	1.34	0.8	2.6	2.06	1.5	4.5	1.06	1.8	3.3	1.21	1.4	3.4	1.10	1.0	3.4	1.28	1.3	2.8	1.38	2.3	3.8	1.40	0.9	3.8	1.48	1.0	3.9
196	0.66	1.3	2.3	0.51	1.8	1.7	0.89	2.8	4.0	1.02	1.7	3.3	2.18	3.7	5.8	2.74	3.6	5.6	1.68	1.8	5.9	1.06	1.6	4.6	0.94	1.5	3.0	0.87	1.5	2.6	0.51	1.1	1.4	0.48	0.7	2.5
197	0.82	0.9	2.5	0.52	0.6	1.5	0.50	0.9	1.7	0.90	2.0	2.7	1.88	2.5	5.7	2.76	3.7	9.3	1.79	3.3	9.7	1.22	1.7	3.1	1.27	2.7	5.0	0.70	1.2	2.8	0.62	1.5	3.8	0.58	1.0	2.8
198	1.32	0.8	2.3	1.05	0.7	2.1	1.06	0.6	3.1	0.82	0.6	1.5	1.71	1.8	4.5	1.98	1.8	2.9	1.15	1.6	3.1	1.01	1.8	2.6	1.47	1.5	3.1	0.94	1.2	3.4	1.54	1.4	5.2	1.14	0.8	4.3
199	0.82	1.8	3.7	0.63	0.8	1.7	0.76	0.8	2.4	1.10	1.6	3.9	2.12	2.0	6.2	2.32	3.7	8.8	1.14	1.8	3.2	0.73	1.1	1.9	1.22	2.2	3.7	0.91	2.2	2.8	0.72	1.8	3.2	0.77	1.0	4.8
200	1.38	1.5	4.8	1.29	1.1	2.9	1.57	1.6	4.3	1.40	1.2	3.3	1.67	1.6	4.0	1.12	1.1	3.2	0.66	0.8	2.8	0.72	1.8	3.1	0.88	1.6	2.6	1.22	0.9	3.2	0.92	0.8	2.8	1.02	1.1	2.6
201	1.91	2.2	5.2	1.42	1.7	6.5	1.56	2.7	7.7	1.22	1.3	4.7	1.47	1.6	4.9	0.86	1.6	3.4	0.38	1.3	2.0	0.21	0.7	1.6	0.47	1.3	2.3	1.06	1.8	4.1	1.40	2.5	4.4	1.68	1.6	6.0
202	1.39	1.4	4.5	1.20	1.0	4.3	1.10	1.1	4.0	0.98	0.7	2.8	1.52	1.1	3.0	1.36	1.3	5.7	0.51	1.9	2.6	0.32	1.4	1.8	0.81	1.5	2.7	1.12	0.9	2.8	1.46	1.1	4.2	1.41	1.1	6.3
203	2.16	1.5	4.5	1.72	1.0	3.9	1.20	0.7	2.8	1.13	1.0	4.0	1.37	1.7	3.4	1.30	2.2	5.1	0.62	1.2	2.4	0.62	1.2	2.1	0.82	1.1	2.5	1.15	1.4	4.8	2.11	1.5	5.8	2.15	0.9	4.0
204	4.74	2.5	9.8	3.65	2.0	8.1	2.60	2.8	6.2	2.30	1.7	5.6	1.84	1.3	4.3	1.32	1.4	3.7	0.58	1.0	2.4	0.66	1.4	3.5	1.65	1.6	3.4	2.72	2.2	6.0	3.34	2.7	9.5	5.30	3.5	11.8
205	10.48	...	29.7	12.48	...	22.2	14.76	...	27.3	8.37	...	22.1	4.92	...	6.1	3.66	...	8.4	1.27	...	3.7	1.92	...	6.5	3.78	...	17.9	12.34	...	25.9	16.46	...	28.5	16.15	...	31.3
206	6.90	4.8	15.8	6.82	3.7	12.5	5.34	2.6	14.1	3.97	2.7	10.9	2.78	1.5	8.2	2.29	1.6	6.1	0.79	0.9	3.3	1.08	1.8	3.2	2.06	3.2	5.0	4.77	2.4	10.1	6.44	3.0	16.6	9.16	3.4	17.3
207	12.16	...	23.8	9.18	...	20.2	8.75	...	21.3	5.42	...	13.4	3.76	...	8.5	3.06	...	7.2	1.21	...	7.6	1.36	...	6.9	3.46	...	8.7	5.65	...	13.4	11.08	...	19.1	12.03	...	24.7
208	6.58	6.9	13.7	5.40	3.8	13.4	4.58	2.4	12.8	2.91	1.9	7.9	2.23	1.8	6.6	1.69	2.1	5.4	0.59	1.8	3.5	0.62	1.7	3.4	1.90	2.9	5.5	3.25	3.0	11.6	6.48	4.4	15.8	6.60	7.7	20.1
209	12.16	...	29.9	9.74	...	24.7	8.14	...	16.5	5.76	...	13.8	2.66	...	7.9	2.88	...	6.8	0.80	...	4.0	0.92	...	4.8	2.82	...	10.8	6.29	...	15.0	12.01	...	26.1	13.06	...	35.1
210	2.01	...	7.1	2.08	...	6.4	1.44	...	6.1	0.69	...	2.0	0.65	...	2.9	0.54	...	2.1	0.21	...	1.4	0.19	...	0.9	0.80	...	5.8	1.09	...	4.8	2.57	...	9.4	3.06	...	10.8
211	1.27	1.1	3.4	1.19	0.7	2.6	1.19	1.0	2.4	0.99	1.1	2.0	1.47	1.2	3.3	1.20	1.5	2.8	0.58	1.2	2.1	0.48	1.3	1.4	0.76	1.8	2.4	0.82	1.1	2.5	1.00	0.8	2.6	1.31	1.4	4.0
212	3.45	3.4	10.4	4.48	4.1	12.2	3.50	1.8	8.6	2.23	3.2	4.2	1.82	1.5	6.5	1.12	1.7	3.8	0.36	1.5	2.2	0.30	1.6	2.2	1.22	1.4	3.8	2.45	2.1	5.8	4.68	3.2	9.9	5.46	3.9	12.8
213	21.14	...	48.8	17.50	...	25.8	10.57	...	17.7	4.58	...	11.3	6.22	...	12.2	3.10	...	9.0	0.29	...	1.6	0.29	...	2.5	2.28	...	3.8	7.88	...	16.2	16.16	...	39.7	18.90	...	35.9
214	11.90	...	25.1	11.42	...	27.8	9.88	...	29.8	5.87	...	12.5	3.81	...	11.4	1.78	...	7.5	0.40	...	4.2	0.34	...	2.0	2.30	...	11.6	4.81	...	14.2	10.08	...	31.9	12.14	...	26.3
215	10.98	...	26.5	7.98	...	18.0	7.44	...	34.8	4.74	...	13.9	1.92	...	6.1	0.85	...	4.2	0.15	...	0.7	0.08	...	1.0	0.82	...	3.0	2.77	...	12.5	7.04	...	24.7	10.52	...	28.6
216	7.58	5.1	18.2	6.80	4.9	19.8	6.78	3.9	19.0	3.29	2.9	11.1	2.07	2.2	7.2	0.88	3.0	4.7	0.14	1.2	1.3	0.18	2.6	2.7	1.04	2.8	4.3	2.33	3.6	8.4	5.38	4.0	14.8	6.37	3.6	12.9
217	7.22	...	19.8	5.69	...	16.2	4.02	...	12.8	3.04	...	8.3	1.68	...	5.4	0.77	...	3.4	0.12	...	0.7	0.10	...	1.4	0.64	...	3.7	2.02	...	8.4	5.00	...	16.6	6.80	...	14.7
218	12.66	...	33.8	9.65	...	29.8	10.01	...	37.5	5.09	...	16.7	3.66	...	10.7	1.24	...	7.1	0.11	...	1.1	0.05	...	0.4	1.10	...	6.9	4.60	...	26.7	6.77	...	29.4	10.10	...	33.2
219	4.75	5.1	20.7	3.88	4.1	16.7	3.35	3.1	12.6	1.68	2.0	7.0	1.12	1.7	3.1	0.46	1.4	2.8	0.05	0.4	0.7	0.04	0.3	0.5	0.83	6.1	7.5	1.37	2.7	8.4	2.82	4.7	17.0	4.42	5.0	12.8
220	3.96	...	14.4	3.11	...	8.7	7.78	...	10.6	1.39	...	7.2	0.82	...	3.6	0.22	...	2.0	Tr.	...	0.1	0.01	...	0.3	0.41	...	3.4	1.03	...	5.9	2.16	...	8.2	3.67	...	12.4
221	30.62	...	71.8	16.80	...	62.2	11.92	...	26.7	4.57	...	13.4	2.96	...	14.9	0.71	...	4.8	0.07	...	1.2	0.01	...	0.1	1.38	...	9.1	3.92	...	15.7	10.20	...	27.6	12.94	...	34.5
222	6.68	...	18.4	4.92	...	14.4	4.36	...	12.9	1.63	...	5.2	1.30	...	4.8	0.22	...	1.3	0.05	...	0.7	0.02	...	0.3	0.57	...	4.4	1.57	...	8.8	3.16	...	9.6	5.57	...	15.9
223	11.30	...	29.7	8.37	...	29.6	7.16	...	26.3	3.50	...	16.6	2.01	...	8.5	0.77	...	4.6	0.10	...	1.3	0.02	...	0.3	0.98	...	6.1	2.72	...	10.9	6.19	...	19.3	8.09	...	23.6
224	4.15	3.1	9.5	3.00	2.2	8.7	2.80	2.1	8.1	1.64	1.8	9.0	0.89	1.8	3.8	0.36	1.4	1.7	0.08	0.4	0.7	0.04	0.9	0.9	0.60	2.8	3.4	1.33	2.5	8.2	2.46	2.3	6.8	3.42	2.2	11.4
225	6.81	2.8	16.0	4.98	3.0	12.7	3.93	2.5	10.4	1.39	1.5	3.6	1.22	1.7	5.1	0.27	1.0	1.4	0.02	0.1	0.1	0.02	0.1	0.2	0.65	2.1	2.9	1.62	1.3	4.3	3.22	3.0	8.7	4.42	1.9	11.7
226	4.90	4.7	24.6	3.76	3.6	12.8	3.09	2.8	8.7	1.58	2.4	10.1	0.72	1.4	6.0	0.14	1.2	2.0	0.03	0.3	0.2	0.02	0.2	0.3	0.32	2.6	5.1	0.99	2.0	7.3	2.54	4.6	11.8	4.48	3.4	15.2
227	3.90	4.6	12.4	2.46	2.6	11.0	2.66	2.6	7.7	1.06	0.8	4.5	0.55	1.2	2.2	0.11	0.4	2.1	Tr.	0.1	0.1	0.02	0.1	0.7	0.36	4.5	6.2	0.72	0.9	4.5	1.50	1.7	7.4	3.64	2.8	10.5
228	14.11	...	39.4	9.40	...	34.0	9.60	...	25.8	2.56	...	8.5	2.06	...	8.1	0.29	...	2.5	0.01	...	0.2	0.05	...	0.9	0.81	...	9.1	2.75	...	10.7	4.58	...	13.1	9.38	...	25.7
229	3.41	...	11.7	2.67	...	11.7	3.04	...	10.6	1.28	...	5.2	0.67	...	3.1	0.11	...	1.3	0.02	...	0.5	0.02	...	0.5	0.20	...	2.7	0.70	...	4.3	1.56	...	6.8	3.07	...	11.5
230	6.02	5.0	18.3	3.88	3.8	11.9	3.52	4.0	12.4	1.41	2.6	5.8	0.58	2.8	4.2	0.06	0.9	3.8	Tr.	Tr.	Tr.	0.03	0.8	0.9	0.27	2.8	3.6	0.92	1.8	9.2	1.60	4.2	12.9	3.80	4.6	13.0
231	4.46	...	17.2	3.74	...	11.7	2.99	...	10.4	1.05	...	5.7	0.47	...	2.8	0.09	...	1.6	0.02	...	0.4	0.02	...	0.6	0.43	...	7.1	0.75	...	8.6	1.38	...	9.8	3.29	...	22.7
232	3.10	4.2	18.2	3.07	5.1	12.4	2.72	3.8	12.4	1.04	3.1	7.8	0.45	1.6	3.6	0.08	0.9	1.4	0.01	0.2	0.3	0.02	0.3	0.3	0.17	1.3	1.3	0.65	3.6	7.0	1.20	3.8	6.8	2.66	4.8	15.8
233	6.08	...	29.9	6.57	...	19.2	5.12	...	15.7	1.40	...	2.7	1.37	...	4.4	0.24	...	1.6	0.02	...	0.3	0.08	...	1.2	0.63	...	4.2	0.37	...	5.0	0.86	...	11.7	3.01	...	19.0
234	3.82	...	24.6	7.96	...	19.4	6.87	...	18.1	1.69	...	4.3	0.85	...	3.5	0.18	...	1.5	0.04	...	0.4	0.12	...	0.4	0.88	...	4.0	1.13	...	6.1	1.67	...	5.4	2.72	...	13.8
235	3.82	...	9.7	2.01	...	7.9	2.12	...	6.9	0.70	...	2.5	0.56	...	2.0	0.04	...	0.5	0.02	...	0.3	0.17	...	3.0	0.13	...	1.3	0.50	...	2.9	0.72	...	2.8	1.64	...	7.8
236	1.88	2.6	7.6	1.94	2.4	9.0	1.50	2.6	7.9	0.67	3.2	5.4	0.32	1.4	2.5	0.06	0.2	0.7	0.05	0.8	1.3	0.09	1.8	1.9	0.09	1.5	1.6	0.42	3.2	3.7	0.89	2.7	3.4	1.88	3.0	9.3
237	3.70	...	20.4	4.28	...	13.3	3.27	...	10.7	1.51	...	5.7	0.78	...	4.4	0.10	...	1.3	0.56	...	3.4	1.05	...	16.1	0.28	...	1.9	0.52	...	2.5	1.54	...	5.8	2.21	...	9.3
238	7.71	...	36.5	7.45	...	24.7	7.56	...	22.4	2.41	...	8.2	1.97	...	8.2	0.24	...	3.0	0.44	...	2.1	0.75	...	3.0	0.53	...	3.0	1.73	...	5.5	3.13	...	12.0	4.86	...	21.0
239	0.32	...	1.3	0.42	...	3.4	0.28	...	3.4	0.08	...	1.4	0.04	...	1.1	Tr.	...	0.1	0.15	...	2.5	0.42	...	5.4	0.08	...	0.9	0.10	...	0.7	0.16	...	1.0	0.32	...	3.2
240	0.62	...	6.0	0.44	...	3.2	0.32	...	3.1	0.07	...	0.7	0.02	...	0.5	0.01	...	0.3	0.04	...	0.4	0.16	...	1.2	0.22	...	2.1	0.14	...	1.9	0.20	...	1.1	0.38	...	3.3

TABLE 1—

Station No.	Station.	Elevation above sea, in feet.	Years of Record.			Maximum Observed Precipitation.					Annual Precipitation.			Observed Precipitation, in Percentages of Mean Annual Rainfall.										Main references.
			For day or less.	For month and year.	Last year included.	Number of Minutes.			Day of 24 hours.	Month.	Minimum.	Mean.	Maximum.	Mean daily intensity on days of rainfall.	Maximum During Short Periods. Number of Minutes.			Mean for month of maximum record.	Maximum.		Annual.			
						5.	15.	60.							5.	15.	60.		In 24 hours.	In one calendar month.	Minimum.	Mean.	Maximum.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
241	San Bernardino, Calif.	1 054	....	49	1919	....	....	....	....	15.5	5.5	16.1	37.1	2.2	....	....	....	22.2	....	96.4	34	100	230	2
242	Squirrel Inn, Calif.	5 280	....	17	1919	....	....	....	16.7	50.3	19.6	39.0	82.2	....	....	....	....	24.9	42.7	128.6	50	100	211	2
243	Barstow, Calif.	2 105	....	24	1919	....	....	....	....	3.9	0.8	4.1	11.0	6.7	....	....	....	13.0	....	95.0	20	100	268	2
244	Mohave, Calif.	2 751	....	38	1914	....	....	....	....	7.2	0.2	4.9	14.8	....	....	....	....	20.2	....	149.0	4	100	302	2
245	Tehachapi, Calif.	3 964	....	39	1914	....	....	....	....	6.3	4.0	10.3	21.4	....	....	....	....	24.0	....	61.0	39	100	207	2
246	Bakersfield, Calif.	404	....	33	1922	....	....	....	....	3.2	3.0	5.7	9.3	....	....	....	....	19.8	....	56.1	53	100	164	2
247	Death Valley, Calif.	—178	....	16	1927	....	....	....	....	1.9	0.4	1.8	4.5	....	....	....	....	17.2	....	105.8	22	100	250	2
248	Independence, Calif.	3 957	....	36	1919	....	....	....	....	12.2	1.1	5.3	21.0+	3.7	....	....	....	21.9	....	230.0	21	100	396+	2
249	Lemon Cove, Calif.	600	....	24	1922	....	....	....	....	15.2	6.2	15.4	30.6	2.4	....	....	....	19.8	....	97.3	40	100	199	2
250	Fresno, Calif.	292	40	47	1927	....	....	....	2.9	9.5	3.9	9.7	21.5	2.2	2.7	5.1	10.0	10.3	29.9	96.2	40	100	222	1; 2
251	Yosemite, Calif.	3 960	....	18	1922	....	....	....	....	24.6	17.3	34.8	69.1	....	....	....	....	21.3	....	70.7	50	100	199	2
252	Merced, Calif.	172	....	51	1922	....	....	....	....	8.0	4.3	11.2	23.8	2.4	....	....	....	21.5	....	71.4	38	100	212	2
253	Lick Observatory, Calif.	4 209	....	42	1922	....	....	....	....	33.8	15.8	30.4	90.1	1.3	....	....	....	19.5	....	111.2	52	100	296	2
254	Stockton, Calif.	23	....	56	1922	....	....	....	....	11.5	6.4	14.4	26.4	....	....	....	....	19.0	....	79.9	44	100	183	2
255	Mokelumne Hill, Calif.	1 550	....	41	1922	....	....	....	....	23.0	15.2	31.4	54.9	1.6	....	....	....	13.4	....	73.4	48	100	175	2
256	Tamarack, Calif.	8 000	....	22	1922	....	....	....	....	29.8	27.8	50.6	94.3	....	....	....	....	22.8	....	78.7	55	100	186	2
257	Placerville, Calif.	1 925	....	49	1923	....	....	....	....	28.8	19.4	43.2	74.1	....	....	....	....	19.6	....	66.8	45	100	172	2
258	Sacramento, Calif.	71	25	79	1927	0.29	0.64	0.94	7.2	15.0	7.8	18.3	34.9	1.7	1.6	3.5	5.2	19.6	38.4	82.0	43	100	190	2
259	Folsom, Calif.	252	....	63	1923	....	....	....	....	21.1	8.5	24.3	46.1	....	....	....	....	20.8	....	86.8	35	100	190	2
260	Auburn, Calif.	1 360	....	54	1923	....	....	....	....	23.1	16.6	33.7	60.6	....	....	....	....	19.7	....	68.5	49	100	180	2
261	Nevada City, Calif.	2 580	....	60	1923	....	....	....	....	41.9	25.7	53.0	102.8	....	....	....	....	18.6	....	79.0	49	100	194	2
262	Lake Spaulding, Calif.	4 600	....	30	1923	....	....	....	....	49.0	36.9	69.8	119.4	....	....	....	....	20.0	....	70.4	53	100	171	2
263	Tahoe, Calif.	6 230	....	15	1923	....	....	....	....	21.2	21.6	31.3	40.6	....	....	....	....	24.0	....	67.5	69	100	130	2
264	Truckee, Calif.	5 819	....	50	1920	....	....	....	....	20.9	6.7	26.2	43.8	....	....	....	....	22.2	....	79.6	26	100	167	2
265	Lewer's Ranch, Nev.	6 282	....	27	1915	....	....	....	....	21.5	10.7	25.2	48.5	....	....	....	....	21.0	....	85.8	42	100	192	2
266	Reno, Nev.	4 532	39	39	1926	....	....	....	2.7	6.8	4.7	8.2	13.7	2.0	....	....	....	20.0	33.0	63.0	57	100	167	2
267	Winnemucca, Nev.	4 344	36	51	1920	....	....	....	1.6	5.2	4.9	8.4	18.4	1.4	....	....	....	11.4	19.0	62.0	58	100	219	2
268	Beowawe, Nev.	4 695	....	51	1920	....	....	....	....	3.7	2.2	6.3	14.9	3.6	....	....	....	9.4	....	58.8	35	100	237	2
269	Clover Valley, Nev.	6 000	....	27	1920	....	....	....	....	8.7	7.5	13.8	24.4	....	....	....	....	15.1	....	63.0	54	100	177	2
270	Wells, N. E., Nev.	5 628	....	46	1915	....	....	....	....	4.7	3.4	9.1	21.6	2.7	....	....	....	14.6	....	51.6	37	100	237	2
271	Kelton, N. W., Utah	4 230	....	43	1920	....	....	....	....	3.7	1.5	6.4	14.5	3.6	....	....	....	10.0	....	57.8	23	100	226	2
272	Corinne, Utah	4 240	....	51	1920	....	....	....	....	5.4	5.4	12.6	22.3	2.4	....	....	....	7.8	....	42.9	43	100	177	2
273	Salt Lake City, Utah	4 408	54	54	1927	....	....	....	2.7	5.8	10.3	16.4	23.7	1.1	....	....	....	12.4	16.4	35.3	63	100	144	2
274	Strawberry Tunnel, Utah	7 606	....	13	1927	....	....	....	....	6.0	12.6	21.5	28.3	1.1	....	....	....	11.5	....	28.1	59	100	132	1
275	Modena, S. W., Utah	5 479	27	27	1927	....	....	....	2.8	4.7	5.1	11.3	19.1	1.5	....	....	....	12.4	20.4	41.6	45	100	169	2
276	St. George, S. W., Utah	2 800	....	38	1927	....	....	....	....	10.2	3.5	8.7	18.7	2.8	....	....	....	13.3	....	117.4	40	100	215	1; 4
277	Alton, S., Utah	7 000	....	19	1920	....	....	....	....	12.0	12.2	22.7	34.8	1.9	....	....	....	17.4	....	53.8	54	100	153	2
278	Moab, S. E., Utah	4 000	....	32	1920	....	....	....	....	6.6	4.3	9.2	16.0	1.9	....	....	....	9.2	....	71.8	47	100	174	2
279	Fort Duchesne, Utah	4 941	....	33	1920	....	....	....	....	5.9	4.4	7.1	12.1	3.5	....	....	....	15.9	....	83.1	62	100	170	2
280	Steamboat Springs, Colo.	6 683	....	32	1922	....	....	....	....	6.0	13.9	22.9	32.1	1.1	....	....	....	7.8	....	21.8	61	100	140	2
281	Grand Junction, Colo.	4 602	19	37	1922	0.39	0.59	0.74	2.5	3.8	5.4	8.4	11.6	1.4	4.6	7.0	8.8	10.2	29.8	45.2	64	100	138	2
282	Durango, S. W., Colo.	6 580	....	37	1922	....	....	....	....	7.4	8.9	20.0	34.3	1.2	....	....	....	9.0	....	37.0	44	100	171	2
283	Wagon Wheel Gap, Colo.	9 610	....	25	1922	....	....	....	....	6.5	7.8	16.1	25.0	1.2	....	....	....	16.0	....	40.4	48	100	155	2
284	Gunnison, Colo.	7 670	....	31	1922	....	....	....	....	4.2	6.3	9.8	13.4	1.5	....	....	....	14.7	....	42.9	63	100	137	2
285	Marshall Pass, Colo.	10 844	....	18	1916	....	....	....	....	6.6	12.5	17.2	21.8	....	....	....	....	11.0	....	28.3	73	100	127	2
286	Leadville, Colo.	10 248	....	35	1922	....	....	....	....	7.7	11.7	16.7	25.8	0.9	....	....	....	13.2	....	48.1	70	100	154	2
287	Long's Peak, Colo.	8 956	....	28	1922	....	....	....	....	7.8	13.9	21.4	29.8	1.2	....	....	....	14.3	....	36.4	65	100	139	2
288	Greeley, Colo.	4 649	....	35	1922	....	....	....	....	6.3	5.4	12.6	20.9	1.8	....	....	....	13.6	....	50.0	43	100	166	2
289	Denver, Colo.	5 292	25 50	56	1927	0.91	1.54	2.20	6.5	8.6	7.7	14.3	23.0	1.2	6.4	10.8	15.4	23.3	45.4	60.0	54	100	161	1
290	Pike's Peak, Colo.	14 111	....	18	1894	....	....	....	....	20.5	9.3	30.0	44.6	0.7	....	....	....	8.6	....	68.3	31	100	149	2
291	Lake Moraine, Colo.	10 265	....	28	1922	....	....	....	....	16.5	18.5	24.7	33.5	1.1	....	....	....	13.1	....	86.9	75	100	136	2
292	Colorado Springs, Colo.	6 098	....	50	1922	....	....	....	....	7.3	8.6	14.4	24.5	1.4	....	....	....	15.6	....	50.8	60	100	170	2
293	Pueblo, Colo.	4 685	35	54	1927	0.45	0.99	2.07	2.9	11.9	6.1	11.5	20.3	1.5	3.9	8.6	18.0	11.4	25.2	103.4	53	100	177	2; 1
294	Las Animas, Colo.	3 899	....	56	1922	....	....	....	....	7.8	3.8	12.3	19.3	2.3	....	....	....	12.5	....	61.3	31	100	157	2
295	Raton, N. Mex.	6 660	....	26	1921	....	....	....	....	11.3	8.0	16.0	23.3	1.9	....	....	....	19.3	....	70.7	50	100	146	2
296	Anchor Mine, N. Mex.	10 600	....	10	1920	....	....	....	....	11.0	28.9	35.8	43.1	0.9	....	....	....	16.1	....	30.7	81	100	120	2
297	Taos Canyon, N. Mex.	8 959	....	13	1921	....	....	....	....	7.7	12.2	21.5	26.8	1.0	....	....	....	15.8	....	35.8	57	100	125	2
298	Fort Union, N. Mex.	6 835	....	62	1918	....	....	....	....	12.6	6.3	18.4	29.2	....	....	....	....	20.6	....	48.5	34	100	159	2
299	Santa Fe, N. Mex.	7 013	57	76	1927	....	....	....	2.5	7.9	5.0	14.1	24.8	1.1	....	....	....	15.4	17.7	56.1	35	100	176	2
300	Albuquerque, N. Mex.	5 100	....	63	1921	....	....	....	....	8.1	3.3	7.8	16.3	4.3	....	....	....	9.7	....	103.9	42	100	209	2
301	San Marcial, N. Mex.	4 447	....	59	1920	....	....	....	....	11.9	1.1	9.3	24.6	3.8	....	....	....	20.3	....	129.3	12	100	265	2
302	Roswell, N. Mex.	3 578	27	27	1920	0.19	0.53	1.40	5.6	9.6	4.9	14.1	22.7	1.8	1.3	3.8	9.9	13.8	39.7	68.2	35	100	161	2
303	Las Cruces, N. Mex.	3 865	....	66	1920	....	....	....	....	5.3	3.5	8.5	17.1	2.4	....	....	....	20.1	....	68.4	41	100	201	2
304	Fort Bayard, N. Mex.	6 152	....	51	1920	....	....	....	....	9.6	5.6	16.5	21.2	1.6	....	....	....	20.6	....	62.0	37	100	[illegible]	2
305	Fort Apache, Ariz.	5 200	....	45	1921	....	....	....	....	9.3	8.7	18.3	33.1	1.4	....	....	....	18.6	....	50.8	48	100	181	2
306	Flagstaff, Ariz.	6 907	....	32	1920	....	....	....	....	12.6	15.7	22.8	34.5	1.2	....	....	....	12.4	12.7	55.3	69	100	151	2
307	Grand Canyon, Ariz.	6 866	....	24	1926	....	....	....	....	8.5	10.7	18.2	36.7	1.7	....	....	....	8.5	....	46.6	59	100	202	2
308	Fort Mohave, Ariz.	604	....	46	1913	....	....	....	....	11.2	1.6	5.1	21.4	6.7	....	....	....	17.3	....	219.5	31	100	420	2; 15
309	Prescott, Ariz.	5 320	....	53	1920	....	....	....	....	8.8	10.0	17.8	30.5	1.6	....	....	....	16.6	....	48.5	56	100	[illegible]	2
310	Phoenix, Ariz.	1 108	32	51	1926	....	....	....	5.0	6.5	3.0	7.8	19.7	2.7	....	....	....	14.0	66.7	86.5	40	100	[illegible]	2
311	Pinal Ranch, Ariz.	4 520	....	29	1921	....	....	....	....	14.9	11.8	25.3	55.4	1.8	....	....	....	14.8	....	58.6	47	100	[illegible]	1; 2
312	Tucson, Ariz.	2 423	....	55	1921	....	....	....	....	6.3	5.2	11.7	24.2	2.2	....	....	....	22.2	....	53.7	45	100	[illegible]	2
313	Yuma, Ariz.	141	57	57	1926	....	....	....	4.0	6.2	0.6	3.4	11.4	6.8	....	....	....	17.0	117.5	187.8	18	100	[illegible]	1
314	Chihuahua, Mexico	4 670	....	13	1918	....	....	....	....	7.6	9.4	15.0	20.4	....	....	....	....	17.4	....	61.0	63	100	136	4
315	Mazatlan, Mexico	12	....	38	1920	....	....	....	....	28.0	10.7	28.4	57.3	....	....	....	....	29.4	....	96.0	38	100	202	4
316	Monterey, Mexico	1 732	....	26	1924	....	....	....	....	17.2	3.4	19.5	37.9	....	....	....	....	22.6	....	88.0	17	100	194	4
317	Leon, Mexico	5 930	....	40	1920	....	....	....	....	16.1	12.4	25.2	36.7	....	....	....	....	18.0	....	64.0	49	100	146	4
318	Mexico City, Mexico	7 400	....	47	1924	....	....	....	....	13.6	13.1	22.8	34.4	....	....	....	....	20.3	....	59.5	57	100	151	4
319	Puebla, Mexico	7 050	....	41	1920	....	....	....	....	14.2	22.2	33.3	50.1	....	....	....	....	20.3	....	42.6	67	100	150	4
320	Oaxaca, Mexico	5 130	....	25	1915	....	....	....	....	16.4	16.7	26.9	44.0	....	....	....	....	22.5	....	61.0	62	100	164	4
321	Merida, S. E., Mexico	72	....	26	1924	....	....	....	....	17.7	16.0	35.1	64.2	....	....	....	....	15.5	....	50.1	46	100	183	4
322	Corosal, Belize	....	....	16	1899	....	....	....	....	17.5	33.7	52.5	66.5	....	....	....	....	12.7	....	33.3	64	100	127	15
323	Belize, Belize	....	....	24	1915	....	....	....	....	30.2	63.8	84.5	130.9	....	....	....	....	12.5	....	35.5	75	100	155	15
324	Setal, Guatemala	2 430	....	17	1913	....	....	....	....	49.9	139.0	193.1	263.6	....	....	....	....	14.7	....	25.9	72	100	136	15
325	Chimax, Guatemala	4 260	....	20	1913	....	....	....	....	27.1	68.9	96.0	116.9	....	....	....	....	14.1	....	28.8	72	100	122	15
326	Magdalena, Guatemala	7 220	....	11	....	....	....	....	....	11.3	....	36.6	51.6	....	....	....	....	15.4	....	30.9	....	100	141	15
327	Guatemala, Guatemala	4 855	....	29	1913	....	....	....	....	22.6	25.0	51.8	71.7	0.7	....	....	....	15.5	....	43.8	48	100	138	15
328	San Francisco, Guatemala	2 430	....	25	....	....	....	....	....	49.0	....	168.8	238.8	....	....	....	....	16.7	....	29.0	....	100	141	15
329	San Salvador, Salvador	2 155 Maxima	.... 	23 9	1920 1920	....	....	....	....	18.0	58.1	68.3	83.4	....	....	....	....	16.6	....	26.3	85	100	122	15
330	Rivas, Nicaragua	150	....	21	1900	....	....	....	....	33.3	31.8	69.8	123.4	0.8	....	....	....	24.9	....	48.5	46	100	177	15
331	Greytown, S. E., Nicaragua	50±	....	6	1900	....	....	....	....	55.4	201.6	254.9	296.9	....	....	....	....	14.3	....	21.8	79	100	117	15
332	Sarapiqui, Costa Rica	2 750	....	6	1903	....	....	....	....	52.2	180.2	209.4	268.5	....	....	....	....	14.3	....	25.0	86	100	128	15
333	San Jose, Costa Rica	3 720	....	34	1904	....	....	....	....	28.7	19.5	70.8	154.3	0.6	....	....	....	16.6	....	40.5	28	100	218	4
334	Balboa, Canal Zone	118 Maxima	16	24 16	1921 1921	....	....	....	7.2	20.3	45.6	69.4	91.4	....	....	....	....	14.8	10.4	29.2	66	100	132	15
335	Gamboa, Canal Zone	....	16	39	1921	....	....	....	6.6	28.4	62.0	90.8	136.2	....	....	....	....	13.0	7.3	31.3	68	100	150	15
336	Colon, Canal Zone	36	16	57	1925	....	....	....	8.5	42.5	86.5	127.3	163.4	....	....	....	....	16.3	6.7	33.4	68	100	128	4; 15
337	Porto Bello, Panama	50±	10	10	1921	....	....	....	10.9	58.2	136.6	160.0	237.3	....	....	....	....	10.5	6.8	36.5	85	100	148	15
338	Cartagena, Colombia	50±	....	12	....	....	....	....	....	....	....	35.6	....	....	....	....	....	....	....	....	....	100	....	15
339	Maracaibo, Venezuela	28	9	11	....	....	....	....	4.4	....	....	20.9	....	....	....	....	....	15.0	21.1	....	....	100	....	15
340	Caracas, Venezuela	3 420	16	35	1925	....	....	....	3.3	8.4	22.6	31.7	47.4	....	....	....	....	13.5	10.4	26.5	71	100	150	4; 15
341	Ciudad Bolivar, Venezuela	125	7	9	....	....	....	....	3.6	....	....	36.1	....	....	....	....	....	6.6	10.2	....	....	100	....	15
342	Calabozo, Venezuela	328	5	7	....	....	....	....	4.1	....	....	47.1	....	....	....	....	....	2.5	8.7	....	....	100	....	15
343	Merida, Venezuela	5 384	9	11	....	....	....	....	3.9	....	....	71.8	....	....	....	....	....	14.1	5.4	....	....	100	....	15
344	Bogota, Colombia	8 700	....	50	1923	....	....	....	....	15.5	25.4	41.6	56.8	....	....	....	....	9.8	....	37.2	61	100	136	4
345	Medellin, Colombia	4 950	....	15	....	....	....	....	....	....	....	58.8	....	....	....	....	....	....	....	....	....	100	....	15
346	Buenaventura, Colombia	50±	....	7	....	....	....	....	....	....	....	280.6	....	....	....	....	....	....	....	....	....	100	....	15
347	Quito, Ecuador	9 350	....	14	1915	....	....	....	....	10.3	38.0	43.1	54.0	....	....	....	....	16.8	....	23.9	88	100	125	15
348	Guayaquil, Ecuador	50±	....	4	1899	....	....	....	....	16.1	29.0	38.6	46.0	....	....	....	....	25.7	....	41.7	75	100	119	15
349	Lima, Peru	520	9	18	....	....	....	....	0.2	....	....	1.9	....	....	....	....	....	21.6	10.5	....	....	100	....	15
350	Cuzco, Peru	11 300	....	12	1921	....	....	....	....	12.7	13.1	31.7	45.6	....	....	....	....	17.8	....	40.0	41	100	144	15
351	Caillioma, Peru	13 000	....	17	....	....	....	....	....	....	....	26.1	....	....	....	....	....	....	....	....	....	100	....	15
352	Arequipa, Peru	8 041	....	37	1926	....	....	....	....	8.1	0.02	4.3	12.0	....	....	....	....	40.5	....	188.2	5	100	279	4
353	La Joya, Peru	4 140	....	9	....	....	....	....	....	....	....	0.2	....	....	....	....	....	....	....	....	....	100	....	15
354	Mollendo, Peru	80	....	10	....	....	....	....	....	....	....	0.8	....	....	....	....	....	....	....	....	....	100	....	15
355	La Paz, Bolivia	11 910	....	5	....	....	....	....	....	....	....	21.2	....	....	....	....	....	....	....	....	....	100	....	15
356	Oruro, Bolivia	12 122	....	4	....	....	....	....	....	....	....	2.1	....	....	....	....	....	....	....	....	....	100	....	15
357	Sucre, Bolivia	9 344	....	32	....	....	....	....	....	....	....	26.2	....	....	....	....	....	....	....	....	....	100	....	15
358	La Serena, Chile	115	....	52	1920	....	....	....	....	9.3	0.3	5.4	16.2	....	....	....	....	16.1	....	172.0	6	100	300	4
359	Santiago, Chile	1 702	....	55	1921	....	....	....	....	13.9	5.0	14.2	32.3	....	....	....	....	22.3	....	98.0	35	100	228	4
360	Punta Galera, Chile	131	....	23	1921	....	....	....	....	27.2	53.3	87.5	137.5	....	....	....	....	14.2	....	31.1	61	100	157	4
361	Valdivia, Chile	49	....	52	1921	....	....	....	....	44.0	71.7	106.3	140.7	....	....	....	....	15.4	....	32.0	67	100	132	4
362	Evangelists Is., Chile	186	....	23	1921	....	....	....	....	19.7	85.5	121.0	137.0	....	....	....	....	9.2	....	16.3	71	100	113	4
363	Punta Arenas, Chile	92	....	34	1921	....	....	....	....	4.4	8.7	15.3	22.2	....	....	....	....	8.2	....	28.8	57	100	145	4
364	South Orkney Is.	23	....	18	1923	....	....	....	....	3.4	11.0	15.9	20.7	....	....	....	....	10.8	....	21.4	69	100	130	6
365	South Georgia Is.	13	....	21	1926	....	....	....	....	13.9	40.4	51.3	66.5	....	....	....	....	9.8	....	27.1	79	100	130	6
366	Stanley, Falkland Is.	6	....	17	1920	....	....	....	....	6.5	20.7	27.0	36.8	....	....	....	....	10.2	....	34.1	77	100	136	4

(Continued.)

Station No.	January: Mean month.	January: Maximum: 24 hours.	January: Maximum: Month.	February: Mean month.	February: Maximum: 24 hours.	February: Maximum: Month.	March: Mean month.	March: Maximum: 24 hours.	March: Maximum: Month.	April: Mean month.	April: Maximum: 24 hours.	April: Maximum: Month.	May: Mean month.	May: Maximum: 24 hours.	May: Maximum: Month.	June: Mean month.	June: Maximum: 24 hours.	June: Maximum: Month.	July: Mean month.	July: Maximum: 24 hours.	July: Maximum: Month.	August: Mean month.	August: Maximum: 24 hours.	August: Maximum: Month.	September: Mean month.	September: Maximum: 24 hours.	September: Maximum: Month.	October: Mean month.	October: Maximum: 24 hours.	October: Maximum: Month.	November: Mean month.	November: Maximum: 24 hours.	November: Maximum: Month.	December: Mean month.	December: Maximum: 24 hours.	December: Maximum: Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE 1.—

Station No.	Station	Elevation above sea, in feet	Years of Record			Maximum Observed Precipitation					Annual Precipitation			Mean daily intensity on days of rainfall	Observed Precipitation, in Percentages of Mean Annual Rainfall									Data references
			For day or less.	For month and year.	Last year included.	Number of Minutes			Day of 24 hours.	Month.	Minimum.	Mean.	Maximum.		Maximum During Short Periods			Mean for month of maximum record.	Maximum		Annual			
						5.	15.	60.							Number of Minutes 5.	15.	60.		In 24 hours.	In one calendar month.	Minimum.	Mean.	Maximum.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
367	Bahia Blanca, Argentine	82	....	65	1924	....	....	....	....	12.4	8.1	21.0	50.6	....	....	....	....	10.7	....	59.0	39	100	241	4
368	Ajo-General Lavalle, Argentine	49	....	67	1924	....	....	....	....	14.9	20.0	34.7	71.1	....	....	....	....	9.3	....	42.9	58	100	205	4
369	Buenos Aires, Argentine	82	....	64	1924	....	....	....	....	21.5	19.6	37.2	79.6	....	....	....	....	11.7	....	57.8	53	100	214	4
370	Cordoba, Argentine	1 329	....	53	1924	....	....	....	....	12.4	17.7	27.7	40.0	....	....	....	....	16.8	....	44.7	64	100	144	4
371	San Juan, W., Argentine	2 130	....	42	1924	....	....	....	....	3.6	0.2	3.7	6.7	....	....	....	....	20.0	....	97.3	6	100	181	4
372	Tucuman, N., Argentine	1 470	....	45	1924	....	....	....	....	15.4	16.5	41.9	57.0	....	....	....	....	18.9	....	36.7	39	100	136	4
373	Asuncion, Paraguay	805	....	29	1923	....	....	....	....	18.6	22.6	51.8	78.5	....	....	....	....	11.9	....	35.9	44	100	151	4
374	Montevideo, Uruguay	96	....	30	1924	....	....	....	....	15.6	22.6	38.8	94.5	....	....	....	....	9.3	....	40.1	58	100	244	4
375	Curityba, Brasil	2 979	....	38	1924	....	....	....	....	16.2	35.5	55.0	75.1	....	....	....	....	10.2	....	29.5	65	100	136	4
376	Alto da Serra, São Paulo, Brasil	2 624	....	33	1924	....	....	....	....	50.1	89.3	141.6	219.0	....	....	....	....	8.6	....	36.5	63	100	155	4
377	Rio de Janeiro, Brasil	201	....	75	1925	....	....	....	....	18.6	23.8	42.2	65.0	....	....	....	....	12.3	....	42.5	67	100	150	4
378	Recife, Pernambuco, Brasil	97	....	48	1922	....	....	....	....	56.7	24.9	65.0	178.5	....	....	....	....	16.1	....	87.0	38	100	275	4
379	Fortaleza, Ceara, Brasil	66	....	72	1920	....	....	....	....	27.2	18.4	56.3	109.4	....	....	....	....	23.9	....	48.4	33	100	194	4
380	Para, near Belem, Brasil	33	....	27	1924	....	....	....	....	30.9	64.1	87.0	116.3	....	....	....	....	16.1	....	35.4	74	100	134	15
381	Cayenne, Guiana	20	....	41	1907 ±	....	....	....	....	....	....	118.5	....	....	....	....	....	....	....	....	....	100	....	15
382	Georgetown, Guiana	6	....	30	1923	....	....	....	....	26.6	53.7	88.7	121.9	....	....	....	....	12.6	....	30.0	60	100	138	4
383	Port of Spain, Trinidad	69	18	59	1920	....	....	....	8.2	17.8	39.8	63.7	92.6	....	....	....	....	13.9	5.0	27.5	62	100	145	4; 8
384	Sangre Grande, Trinidad	....	....	37	....	....	....	....	....	....	....	111.9	....	....	....	....	....	....	....	....	....	100	....	8
385	St. George, Granada	812	13	30	1920	....	....	....	4.3	20.9	50.7	76.1	106.0	....	....	....	....	11.0	5.7	27.5	74	100	139	4; 8
386	Barbados Island	122	7	65	1920	....	....	....	4.6	28.4	27.0	54.8	89.3	....	....	....	....	23.9	8.4	51.8	49	100	163	4; 8
387	Fort de France, Martinique	18	....	58	1923	....	....	....	....	....	....	78.8	....	....	....	....	....	....	....	....	....	100	....	8
388	Morne des Cadets, Martinique	1 676	....	9	1923	....	....	....	....	....	....	130.2	....	....	....	....	....	....	....	....	....	100	....	8
389	Christiansted, Virgin Is.	23	....	47	1925	....	....	....	....	18.7	25.0	46.5	74.0	....	....	....	....	9.1	....	40.3	54	100	159	4
390	St. Thomas, Virgin Is.	27	7	17	1924	....	....	....	5.2	....	....	40.9	....	....	....	....	....	15.0	13.0	....	....	100	....	8
391	San Juan, Porto Rico	82	25	27	1925	....	....	....	10.5	15.4	45.9	60.5	79.0	....	....	....	....	9.5	17.3	25.5	76	100	130	4; 8
392	Toro Negro Dam, Porto Rico	2 275	13	13	1924	....	....	....	8.0	....	....	92.6	....	....	....	....	....	16.4	8.6	....	....	100	....	8
393	Santo Domingo, Haiti	60	10	14	1924	....	....	....	7.6	....	....	55.8	....	....	....	....	....	18.1	13.5	....	....	100	....	8
394	Port Au Prince, Haiti	123	41	49	1925	....	....	....	6.1	19.3	33.2	54.0	76.3	0.8	....	....	....	17.3	11.3	35.7	60	100	142	4; 8
395	Morant Point, Jamaica	8	29	50	1919	....	....	....	12.0	....	....	70.3	....	0.8	....	....	....	11.6	17.1	....	....	100	....	8
396	Kingston, Jamaica	24	30	53	1924	....	....	....	8.3	....	....	32.5	....	1.3	....	....	....	8.6	27.5	....	....	100	....	8
397	Hill Gardens, Jamaica	4 900	29	49	1919	....	....	....	28.7	....	....	105.5	....	....	....	....	....	9.9	27.2	....	....	100	....	8
398	Mooretown, Jamaica	....	15	24	1919	....	....	....	21.0	60.4	....	223.0	330.8	0.6	....	....	....	15.6	9.4	31.4	....	100	148	8
399	Santiago de Cuba	118	16	21	....	....	....	....	14.2	....	....	43.8	....	1.1	....	....	....	18.2	32.7	....	....	100	....	8
400	Isle of Pines, near Cuba	200	10	13	....	....	....	....	6.2	....	....	70.7	....	....	....	....	....	15.8	9.7	....	....	100	....	8
401	Havana, Cuba	161	21	67	1925	....	....	....	10.2	22.6	27.9	48.3	65.2	1.0	....	....	....	4.6	21.4	47.0	58	100	135	4; 8
402	Nassau, Bahama Is.	90	52	58	1920	....	....	....	6.0	23.4	25.5	51.0	79.2	0.8	....	....	....	12.5	18.8	46.0	50	100	156	4; 8
403	Prospect, Bermuda	151	47	55	1924	....	....	....	10.7	21.3	33.6	57.8	87.6	0.6	....	....	....	9.4	18.5	37.0	60	100	152	8; 4
404	Sable Island, Canada	25	....	30	1920	....	....	....	....	17.9	43.4	51.8	67.6	....	....	....	....	7.1	....	34.6	84	100	131	4
405	Charlottetown, Canada	48	....	50	1925	....	....	....	....	10.4	13.5	39.0	56.4	....	....	....	....	10.9	....	26.8	47	100	144	4
406	St. Johns' Newfoundland	125	....	52	1925	....	....	....	....	14.0	42.9	54.4	69.0	....	....	....	....	9.8	....	25.6	79	100	127	4
407	Father Point, Canada	20	....	49	1925	....	....	....	....	8.1	19.1	33.6	47.8	....	....	....	....	12.8	....	24.2	57	100	141	4
408	Montreal, Canada	187	....	52	1925	....	....	....	....	8.6	29.2	40.4	53.0	....	....	....	....	8.6	....	21.3	72	100	131	4
409	Toronto, Canada	379	....	52	1925	....	....	....	....	9.1	24.3	31.2	48.5	....	....	....	....	8.8	....	29.3	78	100	155	4
410	Winnipeg, Canada	760	....	40	1925	....	....	....	....	10.1	12.8	20.0	27.2	....	....	....	....	14.6	....	50.5	60	100	136	4
411	Prince Albert, Canada	1 432	....	41	1925	....	....	....	....	8.0	7.4	15.8	29.9	....	....	....	....	14.8	....	50.8	47	100	189	4
412	Edmonton, Canada	2 158	....	43	1925	....	....	....	....	11.1	8.2	17.3	27.8	....	....	....	....	19.8	....	64.0	47	100	160	4
413	Calgary, Canada	3 389	....	41	1925	....	....	....	....	8.9	7.9	16.1	34.6	....	....	....	....	13.8	....	55.3	48	100	215	4
414	Barkerville, Canada	4 180	....	38	1925	....	....	....	....	8.8	30.8	37.6	49.5	....	....	....	....	8.9	....	24.3	85	100	132	4
415	Victoria, Vancouver, Canada	228	....	35	1925	....	....	....	....	13.0	20.7	30.4	51.0	....	....	....	....	19.0	....	43.0	68	100	168	4
416	Sitka, Alaska	65	46	64	1921	....	....	....	5.0	25.5	56.2	84.8	140.2	0.5	....	....	....	12.0	5.9	30.1	66	100	166	2; 20
417	Juneau, Alaska	84	18	36	1925	0.11	0.18	0.53	5.5	20.5	44.8	81.6	106.5	0.5	1.3	2.2	6.5	8.4	6.7	25.2	55	100	130	2; 4
418	Dawson, N. W., Canada	1 053	....	24	1925	....	....	....	....	4.2	9.2	14.4	17.7	....	....	....	....	8.2	....	29.2	64	100	123	4
419	Eagle, Alaska	838	20	25	1922	....	....	....	1.9	4.7	8.8	10.8	13.8	1.1	....	....	....	18.2	17.6	43.5	58	100	126	2; 4
420	Fairbanks, Alaska	500	17	17	1922	....	....	....	1.4	4.6	8.0	11.7	18.7	1.2	....	....	....	6.4	12.0	39.3	68	100	160	2
421	Tanana, Alaska	223	18	25	1925	....	....	....	2.1	5.9	7.8	12.6	17.7	0.9	....	....	....	18.8	16.7	46.1	62	100	141	2; 4
422	Nome, Alaska	28	15	16	1922	....	....	....	2.2	8.4	9.5	17.4	29.5	0.9	....	....	....	16.5	13.2	48.4	54	100	170	4
423	Kodiak, Alaska	18	14	33	1923	....	....	....	4.8	17.8	43.5	60.7	83.6	....	....	....	....	12.3	7.4	29.5	72	100	138	4; 3
424	St. Paul Island, Alaska	20	....	19	1921	....	....	....	....	9.9	21.0	31.3	47.1	0.5	....	....	....	11.4	....	31.7	67	100	150	2
425	Dutch Harbor, Alaska	16	15	18	1922	....	....	....	4.1	20.2	40.9	62.7	73.1	0.6	....	....	....	13.4	6.6	32.3	65	100	116	2; 4
426	Midway Island	....	9	9	....	....	....	....	6.2	....	....	46.3	....	....	....	....	....	11.2	13.6	....	....	100	....	7
427	Kealia, Kauai, Hawaiian Islands	14	16	19	1918	....	....	....	6.7	24.8	16.7	40.8	62.5	....	....	....	....	16.7	21.3	59.8	41	100	153	2
428	Honolulu, Oahu, Hawaiian Islands	38	23	54	1925	....	....	....	13.8	26.9	11.2	30.9	55.9	0.6	....	....	....	12.6	43.6	31.0	37	100	180	4; 1
429	Luakaha, Oahu, Hawaiian Islands	1 111	16	14	1918	....	....	....	10.7	36.9	96.8	160.3	206.0	0.4	....	....	....	8.8	6.7	23.0	60	100	128	2; 1
430	Kaanapali, Maui, Hawaiian Islands	12	....	20	1918	....	....	....	....	17.8	5.5	20.2	33.5	....	....	....	....	20.9	....	91.4	27	100	166	2
431	Puu Kukui, Maui, Hawaiian Islands	5 000	....	7	1918	....	....	....	....	96.0	232.5	370.1	562.0	....	....	....	....	9.5	....	25.9	63	100	152	2
432	Wailuku Mill, Maui, Hawaiian Islands	175	16	27	1916	....	....	....	6.2	33.9	8.7	37.8	54.5	0.7	....	....	....	14.8	22.2	85.8	21	100	180	2
433	Keanae Valley, Maui, Hawaiian Islands	1 000	16	14	1918	....	....	....	21.9	71.0	120.0	253.6	397.2	0.3	....	....	....	11.4	8.6	27.9	51	100	156	2
434	Kawainui, Hawaii, Hawaiian Islands	4 080	....	12	1918	....	....	....	....	84.2	99.5	276.6	503.7	....	....	....	....	10.0	....	30.5	36	100	182	2
435	Waimea, Hawaii, Hawaiian Islands	2 700	....	28	1918	....	....	....	....	27.3	24.4	43.5	80.1	....	....	....	....	11.8	....	62.8	51	100	184	2
436	Humuula, Hawaii, Hawaiian Islands	6 685	....	8	1918	....	....	....	....	16.3	19.1	32.5	53.4	....	....	....	....	13.1	....	50.6	59	100	166	2
437	Hilo, Hawaii, Hawaiian Islands	40	16	36	1918	....	....	....	14.8	68.6	91.3	139.4	194.2	0.4	....	....	....	10.9	10.6	42.0	65	100	139	2; 1
438	Volcano House, Hawaii, Hawaiian Is.	4 000	....	17	1918	....	....	....	....	41.3	47.6	88.6	133.2	0.7	....	....	....	12.6	....	49.2	57	100	150	2
439	Kapapala Ranch, Hawaii, H. I.	2 150	....	33	1918	....	....	....	....	24.4	23.9	58.1	114.6	....	....	....	....	12.3	....	59.1	41	100	197	2
440	Hilea, Hawaii, Hawaiian Islands	320	....	39	1918	....	....	....	....	25.9	12.5	37.9	74.2	1.3	....	....	....	12.8	....	68.3	33	100	196	2
441	Fanning Island	....	....	13	1917	....	....	....	....	....	47.4	118.2	208.8	....	....	....	....	....	....	....	40	100	177	7
442	Malden Island	28	....	31	1919	....	....	....	....	25.7	3.9	28.6	93.6	1.6	....	....	....	16.6	....	90.0	14	100	327	6
443	Papeiti, Society Island	20	....	35	1909	....	....	....	....	42.4	12.8	55.5	130.8	....	....	....	....	14.8	....	76.8	23	100	235	7
444	Rarotanga, Cook Island	20	....	20	....	....	....	....	....	38.3	46.0	82.7	116.6	....	....	....	....	14.2	....	46.3	56	100	141	7
445	Apia, Samoan Islands	9	29	31	1920	....	....	....	10.2	40.0	68.8	107.2	172.7	....	....	....	....	15.5	9.5	37.3	64	100	161	4; 7
446	Pago Pago, Samoan Islands	....	21	22	....	....	....	....	20.0	....	....	196.2	....	....	....	....	....	7.8	10.2	....	....	100	....	7
447	Delanasau, Fiji Islands	70	10	40	....	....	....	....	14.9	....	....	82.9	....	0.7	....	....	....	17.5	16.1	....	....	100	....	7
448	Wainunu, Fiji Islands	....	....	39	....	....	....	....	....	....	....	149.5	....	....	....	....	....	....	....	....	....	100	....	7
449	Suva, Fiji Islands	44	16	34	....	....	....	....	15.8	37.3	73.0	112.4	169.6	0.4	....	....	....	8.5	14.1	33.2	65	100	151	7
450	Auckland, New Zealand	125	....	71	1923	....	....	....	....	14.1	26.2	42.9	74.1	....	....	....	....	6.9	....	32.1	60	100	169	4
451	Wellington, New Zealand	10	....	62	1923	....	....	....	....	13.9	28.5	48.0	67.7	....	....	....	....	8.5	....	28.9	59	100	141	4
452	Christchurch, New Zealand	25	....	47	1923	....	....	....	....	8.0	13.5	25.2	35.5	....	....	....	....	10.4	....	31.8	54	100	140	4
453	Hokitika, New Zealand	9	....	45	1923	....	....	....	....	20.1	88.2	116.0	154.4	....	....	....	....	8.3	....	25.1	76	100	133	4
454	Noumea, New Caledonia	30	....	55	1914	....	....	....	....	19.1	19.0	42.5	75.8	0.3	....	....	....	13.4	....	44.0	45	100	176	7
455	Col D'Amieu, New Caledonia	1 150	....	5	1914	....	....	....	....	....	....	78.3	....	0.6	....	....	....	....	....	....	....	100	....	7
456	Tulagi, Solomon Islands	7	....	21	1918	....	....	....	....	41.9	65.1	115.4	162.1	....	....	....	....	14.2	....	37.3	54	100	141	7
457	Ocean Island	85	12	12	1916	....	....	....	6.7	....	....	80.8	....	....	....	....	....	11.2	8.3	....	....	100	....	9
458	Nauru, Gilbert Island	26	16	16	1918	....	....	....	7.7	29.6	21.4	84.3	163.3	....	....	....	....	18.1	9.1	35.2	25	100	194	9
459	Jaluit, Marshall Islands	15	17	17	1918	....	....	....	11.1	....	....	158.6	....	....	....	....	....	9.7	7.0	....	....	100	....	9
460	Ujelang Island	30	17	17	1918	....	....	....	11.7	....	....	77.4	....	....	....	....	....	13.4	15.2	....	....	100	....	9
461	Guam Island	67	17	17	1923	....	....	....	10.5	....	....	91.1	....	....	....	....	....	15.6	11.5	....	....	100	....	9
462	Yap, Caroline Islands	104	20	20	....	....	....	....	12.6	46.8	72.9	119.1	153.0	....	....	....	....	13.2	10.6	38.5	61	100	128	9
463	Namatanai, New Ireland	....	7	7	1918	....	....	....	8.8	....	....	110.6	....	....	....	....	....	14.1	8.0	....	....	100	....	9
464	Paniola, New Britain	....	10	10	1918	....	....	....	7.8	25.3	42.2	84.0	133.3	0.6	....	....	....	6.3	6.9	30.0	50	100	158	9
465	Peterhafen, French Islands	....	10	10	1918	....	....	....	11.0	....	....	130.2	....	....	....	....	....	13.5	8.4	....	....	100	....	9
466	Port Moresby, New Guinea	127	....	29	1920	....	....	....	....	18.5	21.6	40.8	73.0	....	....	....	....	18.4	....	45.4	53	100	179	6
466-A	Crohamhurst, Australia	1 400	20	20	1912	....	....	....	35.7	107.6	34.7	80.1	201.7	....	....	....	....	19.7	44.6	134.3	31	100	252	15
467	Brisbane, Australia	137	74	74	1925	....	....	....	18.3	40.4	16.3	45.3	88.3	....	....	....	....	18.8	40.5	89.0	36	100	195	15; 4
468	Sydney, Australia	138	67	85	1924	0.50	1.35	2.90	8.9	76.4	21.5	47.9	82.8	....	1.5	2.8	6.1	11.6	18.6	53.0	45	100	173	4; 20; 1
469	Melbourne, Australia	115	67	70	1925	....	....	....	2.8	7.9	15.6	25.6	44.2	....	....	....	....	9.6	13.7	30.8	61	100	172	20
470	Hobart, Tasmania	177	59	83	1925	....	....	....	5.0	10.3	13.4	23.8	43.4	....	....	....	....	7.7	21.0	42.7	56	100	182	15; 12
471	Adelaide, Australia	140	87	87	1925	....	....	....	5.6	8.6	11.4	21.2	30.9	....	....	....	....	14.9	26.5	40.6	54	100	146	20; 6
471-A	Peechywarrina, Australia	....	....	21	1902	....	....	....	....	4.3	1.2	4.5	8.8	....	....	....	....	12.0	....	95.8	27	100	196	15
472	Alice Springs, Australia	1 926	....	31	1924	....	....	....	....	11.3	4.2	11.1	26.6	....	....	....	....	14.2	....	102.0	38	100	240	4
472-A	Nullarbor, Australia	....	....	24	1911	....	....	....	....	5.9	3.6	9.5	18.9	....	....	....	....	8.0	....	62.1	38	100	199	15
473	Perth, S. W. Australia	197	50	50	1925	....	....	....	8.9	12.8	20.3	34.0	46.7	....	....	....	....	20.4	11.5	37.7	59	100	138	20; 15
474	Carnarvon, W. Australia	....	....	11	....	....	....	....	....	....	....	7.8	....	....	....	....	....	....	....	....	....	100	....	15
475	Darwin, N. W. Australia	97	23	55	1924	....	....	....	11.7	27.9	40.6	61.8	87.2	....	....	....	....	25.8	19.0	45.1	66	100	141	4; 15
476	Kupang, East Indies	49	....	45	1923	....	....	....	....	42.3	30.4	58.2	87.6	....	....	....	....	27.1	....	72.9	52	100	150	4
477	Kajoemas, East Indies	3 050	....	36	1923	....	....	....	....	40.3	70.4	99.4	154.0	....	....	....	....	17.9	....	40.6	71	100	155	4

(Continued.)

Station No.	January.			February.			March.			April.			May.			June.			July.			August.			September.			October.			November.			December.		
	Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.	
		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
367	2.61	...	7.3	2.17	...	7.2	2.04	...	10.7	2.24	...	12.4	1.14	...	5.5	0.91	...	5.4	0.98	...	5.1	0.94	...	5.2	1.57	...	6.4	2.32	...	5.7	2.05	...	8.3	2.09	...	5.4
368	3.01	...	12.3	2.62	...	7.5	2.22	...	8.9	3.22	...	14.9	3.06	...	13.6	2.83	...	8.3	2.30	...	6.5	2.65	...	10.2	2.78	...	12.6	2.95	...	6.6	2.77	...	5.6	2.68	...	6.8
369	3.11	...	22.3	2.08	...	8.0	4.37	...	21.5	3.46	...	14.3	2.87	...	9.6	2.32	...	6.5	2.20	...	6.9	2.48	...	10.9	2.99	...	13.7	3.46	...	10.7	3.15	...	8.5	3.88	...	12.5
370	4.19	...	11.2	4.23	...	10.5	3.48	...	8.5	1.77	...	7.4	1.00	...	7.0	0.29	...	0.9	0.33	...	3.1	0.49	...	3.1	0.89	...	4.5	2.37	...	7.5	3.95	...	10.7	4.63	...	12.4
371	0.74	...	3.6	0.51	...	3.5	0.39	...	1.9	0.09	...	1.2	0.09	...	0.6	0.06	...	0.5	0.14	...	2.0	0.08	...	0.9	0.10	...	0.7	0.20	...	1.9	0.19	...	1.3	0.45	...	1.7
372	7.05	...	15.1	7.09	...	12.8	5.82	...	15.4	2.56	...	8.5	1.06	...	3.7	0.55	...	1.7	0.43	...	1.9	0.47	...	3.4	0.63	...	3.0	2.44	...	6.1	3.73	...	7.8	5.90	...	10.9
373	5.47	...	25.7	5.12	...	15.2	4.25	...	12.0	5.20	...	9.6	4.61	...	13.1	2.72	...	6.9	2.20	...	5.5	1.54	...	5.6	3.11	...	9.0	5.47	...	13.1	5.91	...	13.5	6.18	...	18.6
374	3.66	...	6.7	2.85	...	6.2	3.25	...	10.8	4.47	...	14.6	3.51	...	12.6	3.18	...	14.3	2.49	...	9.6	3.62	...	15.6	3.35	...	7.9	2.57	...	7.6	3.19	...	7.7	3.80	...	10.7
375	6.73	...	24.4	5.82	...	10.7	4.48	...	9.2	3.07	...	7.7	4.10	...	14.7	3.98	...	16.1	2.58	...	7.0	3.27	...	9.1	4.80	...	9.2	5.63	...	16.2	4.96	...	13.5	5.59	...	11.5
376	16.98	...	31.0	15.39	...	43.4	14.45	...	24.1	12.13	...	50.1	8.43	...	17.0	7.72	...	15.9	6.65	...	17.5	7.83	...	20.7	10.75	...	26.7	12.44	...	25.0	14.04	...	35.7	14.84	...	24.6
377	4.96	...	24.0	4.50	...	12.2	5.32	...	18.4	4.17	...	17.9	3.18	...	16.1	2.26	...	12.2	1.68	...	6.4	1.75	...	11.2	2.56	...	6.9	3.26	...	8.1	4.06	...	9.9	5.46	...	12.5
378	3.02	...	10.9	3.77	...	13.1	6.45	...	27.6	8.50	...	32.6	11.20	...	22.0	11.20	...	51.7	10.47	...	56.7	6.38	...	37.6	2.64	...	14.9	1.02	...	4.0	1.06	...	4.5	1.18	...	5.4
379	3.35	...	18.3	6.93	...	21.0	11.77	...	28.6	13.46	...	27.2	9.61	...	26.1	4.72	...	12.4	2.13	...	10.4	1.10	...	6.6	0.67	...	3.0	0.51	...	3.6	0.55	...	2.6	1.54	...	5.4
380	10.90	...	15.9	12.10	...	15.3	24.15	...	30.9	12.60	...	16.8	8.90	...	14.0	5.85	...	7.3	5.06	...	10.1	4.13	...	7.3	3.35	...	5.5	2.56	...	5.5	2.52	...	5.3	5.48	...	10.9
381	14.15	...	...	12.08	...	...	15.20	...	...	15.55	...	...	19.90	...	...	14.80	...	...	6.58	...	...	2.59	...	...	1.10	...	...	1.34	...	...	4.65	...	...	10.55	...	...
382	7.85	...	18.3	4.62	...	13.4	7.16	...	22.7	6.01	...	13.6	11.14	...	23.4	11.68	...	22.0	9.80	...	15.5	6.48	...	14.8	3.08	...	7.6	2.95	...	8.9	6.65	...	20.7	11.14	...	26.6
383	2.75	1.8	11.7	1.52	1.5	6.5	1.82	1.5	7.5	1.84	2.5	8.0	3.64	2.3	11.6	7.95	2.6	16.3	8.85	2.5	17.5	9.63	2.7	17.4	7.37	2.9	12.7	6.65	3.2	14.6	7.00	2.2	15.1	4.66	1.8	11.9
384	7.32	...	...	4.06	...	...	4.67	...	...	4.20	...	...	8.43	...	...	13.75	...	...	13.90	...	...	12.55	...	...	8.71	...	...	9.43	...	...	13.26	...	...	11.83	...	...
385	4.42	1.8	9.8	2.78	1.4	7.2	2.74	1.2	6.8	2.28	1.3	5.9	4.59	3.5	10.8	8.32	2.9	19.4	9.79	3.7	17.8	9.86	3.4	19.1	8.26	3.9	12.6	7.65	4.3	14.2	8.39	4.3	20.9	7.27	2.9	19.2
386	3.37	1.4	10.2	2.22	0.9	12.9	1.92	1.6	5.1	1.65	2.6	6.7	2.69	1.1	12.3	4.34	2.0	15.5	5.13	3.2	14.5	7.41	3.8	17.8	7.77	4.6	22.2	7.63	4.4	23.4	6.48	2.4	20.4	4.23	2.5	11.7
387	4.49	...	...	3.58	...	...	3.16	...	...	3.42	...	...	5.45	...	...	7.35	...	...	9.08	...	...	9.83	...	...	9.47	...	...	8.83	...	...	8.12	...	...	6.17	...	...
388	9.68	...	...	6.49	...	...	6.79	...	...	6.93	...	...	6.29	...	...	11.59	...	...	13.48	...	...	15.24	...	...	11.11	...	...	9.95	...	...	10.57	...	...	12.07	...	...
389	2.29	...	5.2	2.04	...	7.1	1.24	...	3.7	2.97	...	17.2	4.38	...	14.3	4.63	...	13.7	3.47	...	12.9	4.25	...	18.7	5.57	...	15.3	5.88	...	17.6	5.88	...	18.0	3.87	...	12.9
390	2.71	1.4	...	1.87	1.6	...	2.41	2.2	...	1.89	1.9	...	2.66	2.4	...	3.19	1.3	...	3.09	2.7	...	3.38	4.6	...	5.44	2.9	...	6.15	5.3	...	4.99	2.6	...	3.16	1.6	...
391	4.07	3.1	12.4	2.83	4.8	7.8	2.97	2.8	7.2	4.38	6.7	13.2	5.13	4.8	14.0	5.30	5.3	12.2	5.73	4.0	11.0	5.96	6.3	10.4	6.06	6.4	13.7	5.53	3.3	20.2	6.91	5.5	13.3	5.72	10.5	15.4
392	3.97	2.6	...	4.79	3.1	...	4.01	2.6	...	7.96	7.6	...	8.64	3.6	...	6.53	4.1	...	7.18	5.1	...	6.70	6.0	...	12.78	5.1	...	15.22	8.0	...	9.88	6.1	...	4.95	3.8	...
393	2.07	1.5	...	1.20	0.6	...	2.75	2.8	...	3.68	6.2	...	5.06	5.0	...	5.76	2.4	...	7.22	4.8	...	6.67	5.4	...	7.22	5.1	...	7.29	7.4	...	4.25	2.5	...	2.29	1.0	...
394	1.22	2.9	6.4	2.51	1.6	10.9	3.69	3.5	12.4	6.48	3.9	14.2	9.35	4.3	19.3	4.10	2.9	13.6	2.69	2.4	5.7	5.37	4.6	10.7	7.28	5.8	11.7	6.57	3.6	14.6	3.44	5.1	13.1	1.26	1.6	4.9
395	4.26	3.3	...	2.89	3.3	...	2.15	4.7	...	3.53	5.8	...	9.35	5.9	...	6.21	8.3	...	3.40	4.9	...	5.25	5.8	...	7.02	5.9	...	11.66	10.2	...	8.24	12.0	...	6.11	6.0	...
396	0.92	1.5	...	0.67	1.8	...	0.96	1.3	...	1.18	1.8	...	4.16	6.8	...	4.00	6.3	...	1.53	4.4	...	3.37	3.1	...	4.42	5.4	...	6.96	6.5	...	2.80	8.9	...	1.55	3.6	...
397	6.98	5.5	...	4.07	6.8	...	4.88	5.0	...	5.98	4.4	...	10.43	28.7	...	7.62	8.4	...	3.64	4.0	...	8.28	26.2	...	9.84	11.2	...	17.20	23.8	...	16.16	18.2	...	10.30	10.7	...
398	17.48	10.7	...	12.92	11.0	...	9.78	10.0	...	11.76	10.6	...	18.97	15.4	60+	21.77	13.1	...	17.72	10.1	...	15.54	7.6	...	15.39	10.6	...	19.92	11.1	...	34.65	21.0	49.4	26.08	11.5	...
399	1.47	3.4	...	0.87	1.0	...	1.72	1.6	...	3.32	3.9	...	6.45	4.5	...	5.39	7.3	...	2.26	6.2	...	3.58	4.4	...	6.66	14.3	...	7.40	8.6	...	3.61	5.1	...	1.06	2.1	...
400	1.67	1.8	...	2.16	3.5	...	2.40	3.5	...	3.93	4.5	...	8.63	3.6	...	11.96	4.5	...	7.58	2.5	...	7.67	3.6	...	11.06	6.8	...	9.93	6.4	...	2.30	3.8	...	1.59	2.8	...
401	2.90	5.7	21.8	1.81	3.1	6.2	1.89	1.9	9.6	2.21	3.7	22.6	4.60	5.5	18.7	5.36	3.8	17.6	4.85	2.8	18.1	5.39	3.1	10.6	5.82	3.4	16.0	6.63	10.3	15.9	3.13	7.0	10.3	2.40	2.5	16.4
402	2.16	4.5	9.7	1.61	2.7	7.0	1.53	3.5	5.3	2.51	2.9	9.9	5.89	7.4	17.8	6.66	5.4	15.3	5.83	4.4	14.4	6.59	6.4	20.1	7.55	6.5	20.5	6.34	8.0	23.4	2.85	7.4	10.4	1.49	3.4	6.0
403	4.48	3.8	9.7	4.60	3.3	10.4	4.74	5.9	10.0	4.10	10.7	13.3	4.54	5.5	13.1	4.49	8.0	11.0	4.53	3.3	18.2	5.40	3.1	21.3	5.19	6.6	16.3	5.95	5.0	17.7	5.01	6.0	11.4	4.78	4.6	10.6
404	5.38	...	8.5	4.60	...	8.2	4.38	...	8.5	3.54	...	5.6	3.40	...	6.5	3.81	...	10.2	3.63	...	11.3	3.65	...	17.9	3.37	...	5.9	4.92	...	13.9	5.65	...	8.7	5.51	...	9.7
405	3.07	...	7.6	2.89	...	6.4	3.13	...	6.2	2.80	...	6.1	2.63	...	5.8	2.65	...	5.4	3.03	...	9.0	2.62	...	6.5	3.31	...	8.7	4.25	...	10.4	3.72	...	8.0	3.70	...	7.2
406	5.37	...	11.4	4.96	...	12.8	4.57	...	8.8	4.27	...	8.8	3.62	...	7.1	3.61	...	7.8	3.75	...	7.7	3.72	...	9.8	3.76	...	11.4	5.35	...	13.1	6.07	...	12.3	5.32	...	14.0
407	2.88	...	4.7	2.14	...	5.1	2.53	...	5.1	1.98	...	4.0	2.91	...	6.7	3.20	...	8.1	3.09	...	5.9	3.11	...	6.2	3.03	...	5.9	4.61	...	8.1	2.87	...	6.0	2.78	...	6.7
408	3.67	...	6.2	3.16	...	6.3	2.53	...	8.5	2.55	...	6.1	3.05	...	6.2	3.49	...	8.6	3.76	...	7.7	3.31	...	7.9	3.52	...	7.8	3.31	...	7.5	3.40	...	7.6	3.61	...	6.5
409	2.76	...	5.5	2.37	...	5.0	2.41	...	5.7	2.36	...	5.4	2.76	...	9.1	2.70	...	5.8	2.86	...	5.6	2.78	...	8.1	2.70	...	7.7	2.51	...	5.4	2.88	...	5.6	2.44	...	4.9
410	0.82	...	3.4	0.78	...	2.5	1.16	...	2.9	1.42	...	5.6	1.94	...	6.4	2.97	...	10.1	3.11	...	7.1	2.10	...	4.7	2.29	...	5.5	1.35	...	5.7	1.07	...	3.0	0.91	...	4.0
411	0.78	...	2.0	0.62	...	2.1	0.89	...	2.7	0.91	...	3.4	1.52	...	4.9	2.74	...	4.6	2.23	...	5.3	2.35	...	8.0	1.42	...	3.3	0.82	...	1.9	0.90	...	3.1	0.66	...	2.1
412	0.88	...	2.3	0.69	...	2.3	0.73	...	1.7	0.85	...	2.6	1.76	...	7.7	3.13	...	8.5	3.39	...	11.1	2.43	...	6.4	1.32	...	4.3	0.73	...	2.3	0.67	...	2.6	0.76	...	3.2
413	0.52	...	1.3	0.55	...	1.9	0.82	...	2.4	0.84	...	2.2	2.22	...	8.9	3.04	...	8.8	2.56	...	5.5	2.44	...	7.7	1.25	...	4.0	0.61	...	2.2	0.69	...	2.7	0.82	...	2.2
414	3.34	...	9.3	2.62	...	5.2	2.62	...	6.5	2.20	...	5.4	2.52	...	4.3	3.49	...	5.9	3.23	...	7.9	3.20	...	8.5	2.67	...	7.7	3.15	...	6.3	3.43	...	5.3	3.53	...	6.7
415	4.54	...	8.5	3.29	...	7.0	2.43	...	5.4	1.52	...	5.4	1.13	...	2.8	0.84	...	2.4	0.42	...	1.2	0.64	...	1.8	1.80	...	4.3	2.57	...	5.6	5.42	...	11.5	5.74	...	13.0
416	7.62	3.3	16.0	6.50	2.5	18.8	5.62	2.5	16.0	5.48	3.3	13.4	4.08	2.1	10.3	3.37	3.0	7.1	4.17	3.1	10.9	7.13	4.3	21.0	10.15	5.0	25.5	12.17	4.9	24.8	9.53	4.0	20.5	8.99	4.2	20.2
417	6.88	4.2	20.5	5.24	2.4	13.4	5.19	1.9	9.6	5.34	2.6	11.4	5.22	2.4	13.1	3.70	1.7	11.5	4.96	2.0	10.5	7.40	3.9	14.0	10.83	5.5	27.1	10.37	3.5	18.6	8.40	4.0	18.5	7.38	2.7	13.7
418	0.82	...	2.0	0.76	...	1.6	0.54	...	1.8	0.62	...	1.7	0.90	...	2.0	1.28	...	2.7	1.58	...	3.3	1.47	...	2.5	1.63	...	3.5	1.22	...	4.1	1.10	...	3.7	1.18	...	4.2
419	0.46	0.4	1.4	0.35	0.5	1.2	0.44	1.0	2.2	0.41	0.6	1.0	0.82	1.0	2.9	1.48	1.0	3.3	1.78	0.9	2.6	1.96	1.9	4.7	1.27	1.0	3.4	0.82	0.8	3.0	0.49	0.5	1.3	0.48	0.7	1.1
420	0.76	0.7	2.2	0.43	0.6	1.2	0.95	0.8	2.8	0.31	0.4	1.0	0.51	0.8	1.4	1.44	1.4	3.2	1.85	0.8	4.1	1.90	1.0	3.7	1.29	1.4	3.6	0.82	0.8	2.6	0.75	0.5	4.6	0.73	0.6	1.5
421	0.68	0.4	3.2	0.76	1.1	3.4	0.67	0.9	2.9	0.25	0.4	1.0	0.82	0.9	1.6	0.97	1.1	2.2	2.36	1.1	5.8	2.06	2.1	3.8	1.67	1.5	5.7	1.12	0.8	4.4	0.65	0.6	3.4	0.71	0.9	2.7
422	0.97	0.9	2.9	1.06	1.6	4.4	0.89	0.9	3.4	0.61	0.5	1.5	0.88	0.6	2.5	1.22	2.3	4.4	2.87	1.7	8.4	3.00	1.1	5.4	2.34	2.1	4.1	1.47	1.1	3.4	0.98	1.3	3.9	1.13	0.8	3.4
423	4.72	1.5	12.9	4.58	1.7	9.0	3.86	1.4	10.6	3.93	2.1	7.8	5.42	2.3	14.6	4.63	2.3	11.2	3.36	1.9	8.6	5.41	4.5	10.3	5.49	2.4	11.8	7.35	1.9	17.8	5.71	1.9	14.1	6.33	2.1	16.3
424	2.56	...	7.7	1.99	...	5.8	2.38	...	4.2	1.69	...	4.6	1.76	...	4.1	1.51	...	5.4	3.12	...	5.3	3.51	...	5.7	3.46	...	8.1	3.55	...	9.9	3.18	...	9.3	2.66	...	6.6
425	5.39	2.4	8.8	7.06	3.3	15.1	5.62	3.2	12.2	3.39	2.0	6.1	5.04	2.6	7.5	2.71	1.9	7.1	2.31	1.3	5.9	3.01	2.1	5.6	5.77	2.6	18.0	8.40	4.1	20.2	6.77	3.6	13.8	7.23	3.0	15.6
426	4.19	3.4	...	3.66	1.7	...	4.24	3.7	...	5.08	5.8	...	3.60	6.1	...	3.35	4.0	...	3.08	4.1	...	3.54	3.1	...	5.01	5.8	...	5.20	6.3	...	1.74	1.4	...	3.01	2.0	...
427	4.00	5.8	15.7	3.06	2.0	21.4	6.81	4.5	24.3	2.50	1.9	8.8	2.43	2.6	7.8	1.92	1.8	3.9	2.00	1.8	4.3	1.97	1.4	4.5	3.02	8.3	18.4	3.16	8.0	9.3	3.95	8.7	21.1	5.06	4.2	14.8
428	2.82	4.5	18.4	3.38	3.5	24.9	3.71	13.5	15.9	2.37	8.0	12.6	1.76	2.0	11.3	1.03	2.0	4.3	1.20	1.2	6.0	1.37	1.1	4.5	1.44	2.7	6.4	1.98	4.6	6.9	3.87	4.3	15.7	4.38	4.8	16.7
429	12.37	6.6	24.2	9.98	10.1	23.8	15.49	10.7	36.1	15.59	8.6	34.3	16.01	6.3	28.0	11.25	4.5	17.9	11.06	4.9	22.9	13.45	6.0	20.2	13.26	7.6	36.9	10.15	4.3	18.7	14.71	5.3	25.4	16.82	4.6	27.2
430	4.72	...	17.5	3.24	...	10.0	2.20	...	6.4	1.42	...	6.0	0.92	...	5.1	0.30	...	1.4	0.48	...	1.9	1.22	...	5.7	0.50	...	8.1	0.89	...	2.2	1.35	...	6.1	3.24	...	7.4
431	25.81	...	50.0	22.52	...	54.0	32.61	...	80.0	46.91	...	92.0	26.00	...	45.0	33.51	...	38.6	35.09	...	96.0	28.80	...	81.6	22.20	...	54.0	24.93	...	38.0	28.21	...	55.0	32.48	...	55.0
432	6.16	6.6	22.9	4.86	3.3	23.0	3.91	3.3	18.0	2.15	4.2	8.5	1.52	2.4	6.8	0.41	0.9	2.0	0.62	0.7	1.7	1.03	1.5	3.1	0.89	1.5	2.4	1.42	1.3	4.9	3.02	3.6	21.0	3.88	6.2	15.6
433	14.97	6.7	29.9	17.04	11.6	37.5	21.22	21.9	63.1	28.82	15.8	71.0	21.09	11.2	59.8	19.87	5.6	35.7	20.80	7.9	44.7	25.78	10.8	56.1	19.61	7.3	45.8	15.91	6.8	32.7	22.18	7.9	43.5	26.27	17.2	59.4
434	18.17	...	40.9	19.58	...	36.6	19.76	...	42.3	24.33	...	49.2	24.96	...	42.5	28.67	...	53.3	30.30	...	62.3	27.60	...	84.2	20.02	...	55.2	17.97	...	43.9	23.95	...	66.6	21.42	...	49.4
435	4.76	...	16.5	4.64	...	15.1	4.98	...	27.3	3.66	...	9.8	3.14	...	9.8	2.45	...	5.2	2.89	...	7.4	3.25	...	10.3	2.19	...	7.0	2.61	...	8.0	3.52	...	7.9	5.61	...	20.8
436	2.48	...	6.2	2.89	...	9.3	2.39	...	4.2	2.12	...	5.5	1.85	...	3.5	1.00	...	2.4	1.79	...	3.9	4.73	...	15.4	2.08	...	5.5	2.59	...	8.8	4.22	...	16.3	4.15	...	14.8
437	9.91	5.5	38.6	11.47	7.0	32.5	15.14	8.2	58.6	13.44	14.5	28.8	10.08	5.6	21.5	8.11	7.4	28.5	10.77	5.1	23.4	12.32	7.3	37.8	10.81	13.6	26.3	11.05	6.8	27.0	14.64	7.5	38.8	11.67	5.0	27.8
438	8.62	...	17.3	7.22	...	24.6	8.41	...	22.2	7.53	...	20.3	5.58	...	10.7	3.88	...	10.2	5.87	...	28.8	7.62	...	23.7	5.82	...	17.8	6.38	...	13.7	11.30	...	41.3	7.07	...	14.3
439	6.28	...	23.0	6.94	...	31.3	8.15	...	29.0	4.00	...	15.8	3.61	...	14.7	1.71	...	7.7	2.76	...	15.9	3.58	...	12.7	3.66	...	10.4	4.31	...	13.1	7.19	...	34.4	5.98	...	21.0
440	3.72	...	16.0	4.12	...	19.7	5.06	...	20.6	2.82	...	12.0	1.79	...	8.6	1.04	...	3.9	1.75	...	8.2	2.81	...	11.9	2.03	...	6.3	2.95	...	11.0	6.22	...	33.9	4.38	...	19.9
441	10.77	...	...	8.72	...	...	12.54	...	...	17.04	...	...	13.99	...	...	11.49	...	...	11.24	...	...	5.71	...	...	3.99	...	...	4.98	...	...	3.76	...	...	12.98	...	...
442	4.06	...	19.5	2.00	...	14.8	4.74	...	25.7	4.63	...	19.1	4.24	...	18.4	1.94	...	12.5	2.01	...	10.1	1.63	...	8.6	0.85	...	3.0	0.97	...	9.6	0.75	...	8.7	0.80	...	8.2
443	8.21	...	...	7.44	...	...	6.94	...	...	4.34	...	...	3.23	...	...	2.72	...	...	1.91	...	...	1.63	...	...	2.29	...	...	3.52	...	...	5.18	...	...	8.01	...	...
444	9.80	...	...	10.85	...	...	11.80	...	...	8.25	...	...	6.43	...	...	4.13	...	...	3.66	...	...	4.55	...	...	4.54	...	...	4.75	...	...	6.49	...	...	7.47	...	...
445	16.66	6.5	40.0	15.53	5.7	30.1	13.41	5.6	24.3	10.16	9.6	20.0	5.54	4.8	17.7	5.14	4.8	12.4	2.72	2.1	6.4	3.50	8.1	15.4	4.99	3.6	14.4	6.32	4.3	12.4	9.60	3.8	20.3	13.89	10.2	28.4
446	21.92	10.2	...	23.50	13.5	...	19.06	7.8	...	17.67	8.6	...	15.27	20.0	...	14.38	16.5	...	9.90	10.6	...	7.66	16.0	...	12.87	13.9	...	15.76	7.5	...	19.33	16.8	...	18.93	12.0	...
447	16.97	12.1	...	14.18	5.6	...	16.20	14.9	...	9.21	4.7	...	4.58	3.5	...	2.37	3.9	...	1.76	3.5	...	2.81	2.7	...	3.38	3.6	...	5.35	2.8	...	6.57	5.5	...	9.58	2.9	...
448	18.98	...	...	17.48	...	...	18.49	...	...	15.27	...	...	11.47	...	...	6.62	...	...	5.67	...	...	6.80	...	...	9.27	...	...	11.16	...	...	12.11	...	...	16.11	...	...
449	10.72	4.3	...	10.13	7.4	...	14.70	9.5	...	11.28	6.9	...	10.16	7.4	...	6.15	5.3	...	4.60	5.9	...	8.24	6.3	...	6.96	5.2	...	7.80	8.4	...	9.51	16.9	...	12.11	10.9	...
450	2.68	...	7.4	3.03	...	14.1	3.04	...	8.7	3.38	...	9.9	4.41	...	10.7	4.65	...	11.2	5.02	...	10.1	4.20	...	10.6	3.63	...	7.4	3.63	...	10.2	3.28	...	7.7	2.85	...	8.8
451	3.33	...	10.1	3.14	...	8.9	3.33	...	8.3	3.88	...	12.1	4.68	...	11.8	4.77	...	9.5	3.63	...	12.2	4.49	...	9.9	3.97	...	11.0	4.08	...	12.9	3.52	...	10.0	3.22	...	12.5
452	2.22	...	8.5	1.83	...	5.7	2.08	...	6.5	1.96	...	7.0	2.62	...	8.0	2.04	...	7.5	2.78	...	6.9	1.76	...	4.4	1.79	...	5.5	1.63	...	6.2	1.86	...	6.8	2.04	...	8.0
453	9.82	...	20.5	7.26	...	17.9	9.69	...	29.1	9.13	...	21.3	9.82	...	22.1	9.68	...	21.6	8.95	...	17.3	9.39	...	17.9	9.20	...	18.7	11.80	...	26.7	10.61	...	20.1	10.62	...	22.4
454	3.78	...	...	4.77	...	...	5.81	...	...	5.13	...	...	4.50	...	...	3.63	...	...	3.09	...	...	2.65	...	...	2.46	...	...	2.06	...	...	2.35	...	...	2.84	...	...
455	11.94	...	...	18.14	...	...	10.54	...	...	7.33	...	...	6.44	...	...	2.94	...	...	3.49	...	...	4.22	...	...	2.41	...	...	1.84	...	...	2.74	...	...	6.28	...	...
456	18.44	...	...	16.18	...	...	16.45	...	...	8.34	...	...	7.50	...	...	5.06	...	...	6.26	...	...	6.86	...	...	7.74	...	...	7.91	...	...	8.48	...	...	10.65	...	...
457	12.88	4.5	...	9.03	6.7	...	8.42	5.0	...	8.49	4.8	...	6.07	5.3	...	8.12	4.0	...	5.67	6.7	...	2.67	2.3	...	8.90	3.3	...	4.80	3.3	...	5.74	4.0	...	8.08	4.5	...
458	8.64	4.6	...	11.06	7.7	...	6.46	6.0	...	5.49	3.9	...	5.72	4.7	...	4.96	3.0	...	6.74	4.0	...	6.08	4.0	...	6.51	4.8	...	5.27	4.4	...	6.78	6.5	...	10.46	5.6	...
459	10.19	6.8	...	8.53	3.2	...	14.22	6.0	...	15.76	8.1	...	16.57	4.1	...	15.27	6.9	...	15.36	11.1	...	12.04	6.0	...	13.05	4.7	...	12.17	7.3	...	11.88	3.7	...	13.57	7.6	...
460	2.12	2.4	...	1.78	2.1	...	2.62	6.3	...	6.25	5.7	...	6.63	3.4	...	7.14	7.0	...	8.36	5.7	...	8.46	4.8	...	10.29	5.0	...	10.28	3.2	...	9.55	11.7	...	4.88	3.9	...
461	2.42	1.5	...	2.86	3.4	...	3.19	6.5	...	2.18	2.3	...	4.07	4.5	...	8.63	2.9	...	14.24	10.5	...	15.70	6.7	...	16.14	6.0	...	12.61	5.7	...	7.30	6.8	...	4.80	3.0	...
462	6.35	6.2	...	7.29	7.1	...	6.03	5.5	...	5.13	3.4	...	10.13	6.8	...	10.15	6.5	...	15.72	12.6	...	15.76	5.1	...	12.02	4.0	...	11.86	4.6	...	9.46	5.9	...	8.17	5.8	...
463	15.86	5.8	...	7.97	3.3	...	17.54	4.8	...	11.82	3.4	...	5.65	2.9	...	7.84	3.5	...	5.99	4.6	...	5.06	2.2	...	4.72	2.0	...	6.07	3.8	...	9.67	3.1	...	12.54	6.3	...
464	9.80	6.5	...	8.16	4.6	...	11.56	5.3	...	7.24	3.5	...	6.03	3.9	...	4.30	3.9	...	5.76	7.2	...	6.04	3.7	...	5.87	7.5	...	4.60	2.7	...	6.15	3.5	...	10.39	6.6	...
465	15.62	7.8	...	11.94	4.7	...	17.91	7.8	...	13.80	4.7	...	10.27	4.8	...	6.08	4.5	...	6.49	3.9	...	4.22	2.8	...	5.70	2.8	...	6.92	2.5	...	13.48	5.5	...	17.62	11.0	...
466	7.48	...	18.5	9.24	...	18.3	6.82	...	15.9	3.30	...	9.7	3.00	...	23.2	0.79	...	1.9	0.98	...	6.4	0.76	...	6.9	1.38	...	6.3	0.82	...	4.9	2.04	...	8.3	4.22	...	17.4
466-A	13.56	29.8	34.4	15.85	35.7	107.4	13.77	16.0	50.8	4.99	...	17.2	5.00	...	24.1	4.40	13.3	18.2	2.88	...	8.7	2.41	...	7.0	2.14	...	8.8	3.09	...	7.5	4.50	...	17.5	6.92	12.8	25.5
467	6.33	18.7	27.7	6.28	8.4	40.4	5.81	11.2	34.0	3.68	4.5	15.3	2.87	5.6	13.6	2.73	6.0	14.0	2.32	3.5	8.5	2.12	4.9	14.7	2.06	3.5	5.4	2.84	1.8	10.0	3.73	4.5	12.4	4.90	6.6	14.0
468	3.72	7.1	15.3	4.26	8.9	18.6	4.79	6.5	18.7	5.57	7.5	25.4	5.13	6.4	23.0	4.77	5.2	16.3	4.84	5.7	13.2	3.01	5.8	14.9	2.80	5.7	14.0	3.21	3.4	24.7	2.81	4.2	11.1	2.91	4.7	13.8
469	1.88	2.0	5.7	1.73	3.4	6.2	2.20	3.5	7.5	2.40	2.3	6.7	2.17	1.8	4.3	3.07	1.7	4.5	1.85	2.7	7.0	1.88	1.9	4.0	2.45	3.6	7.9	2.64	3.0	7.6	2.28	2.6	6.7	2.31	2.6	7.2
470	1.85	3.0	5.9	1.46	4.5	9.1	1.71	2.8	7.6	1.86	5.0	6.5	1.88	2.2	6.4	2.22	4.1	8.1	2.18	2.5	6.0	1.84	4.3	10.2	2.09	3.5	7.1	2.24	2.6	6.7	2.50	4.0	5.9	1.98	2.6	9.0
471	0.73	2.3	4.0	0.72	5.6	6.1	1.04	3.5	4.6	1.75	3.1	6.8	2.76	2.7	7.8	3.14	2.1	8.6	2.64	1.7	6.4	2.50	2.2	6.2	2.04	1.6	5.3	1.73	2.2	3.8	1.15	1.9	3.6	0.99	2.4	6.0
471-A	0.74	...	4.0	0.54	...	4.3	0.31	...	2.0	0.40	...	1.6	0.39	...	2.7	0.60	...	2.6	0.16	...	1.1	0.28	...	1.3	0.21	...	0.9	0.16	...	0.8	0.37	...	2.2	0.42	...	3.1
472	1.76	...	11.1	1.67	...	9.3	1.18	...	8.9	0.75	...	4.6	0.66	...	4.3	0.60	...	2.9	0.40	...	4.2	0.60	...	6.2	0.38	...	3.5	0.74	...	4.8	1.01	...	5.8	1.65	...	11.2
472-A	0.45	...	2.1	0.35	...	1.9	0.76	...	5.9	0.66	...	2.3	1.40	...	4.8	1.54	...	4.2	0.99	...	2.6	1.04	...	2.9	0.66	...	1.9	0.57	...	1.5	0.62	...	2.0	0.49	...	2.6
473	0.84	1.7	3.2	0.45	1.6	3.0	0.78	2.1	4.5	1.58	2.6	5.0	4.94	2.8	12.1	6.92	3.9	12.5	6.45	3.0	11.3	6.62	2.8	10.3	3.40	1.7	7.5	2.10	1.4	7.9	0.79	1.1	2.8	0.58	1.7	3.0
474	0.16	...	...	0.35	...	...	0.82	...	...	0.71	...	...	0.87	...	...	2.09	...	...	2.17	...	...	0.58	...	...	0.24	...	...	0.04	...	...	0.08	...	...	0.00	...	...
475	10.92	11.7	37.9	12.95	5.1	22.6	10.07	6.1	21.9	4.00	3.6	23.7	0.71	1.2	10.3	0.14	1.3	1.5	0.06	1.7	2.6	0.08	1.1	3.0	0.49	2.0	2.3	2.16	3.6	6.3	4.83	2.8	14.6	10.30	7.9	22.4
476	15.70	...	42.3	14.92	...	37.2	8.74	...	26.3	2.52	...	13.2	1.27	...	10.4	0.43	...	2.7	0.20	...	3.0	0.12	...	1.1	0.08	...	1.3	0.79	...	0.7	3.42	...	21.0	9.96	...	21.4
477	15.25	...	28.8	17.74	...	28.8	17.74	...	40.3	8.75	...	24.3	5.95	...	18.7	2.68	...	15.2	1.73	...	15.8	0.67	...	2.7	0.38	...	3.8	1.85	...	12.0	7.92	...	24.7	15.66	...	26.8

TABLE I.—

Station No. (1)	Station. (2)	Elevation above sea, in feet. (3)	Years of Record. For day or less. (4)	Years of Record. For month and year. (5)	Years of Record. Last year included. (6)	Maximum Observed Precipitation. Number of Minutes. 5. (7)	Maximum Observed Precipitation. Number of Minutes. 15. (8)	Maximum Observed Precipitation. Number of Minutes. 60. (9)	Maximum Observed Precipitation. Day of 24 hours. (10)	Maximum Observed Precipitation. Month. (11)	Annual Precipitation. Minimum. (12)	Annual Precipitation. Mean. (13)	Annual Precipitation. Maximum. (14)	Observed Precipitation, in Percentages of Mean Annual Rainfall. Mean daily intensity on days of rainfall. (15)	Maximum During Short Periods. Number of Minutes. 5. (16)	Maximum During Short Periods. Number of Minutes. 15. (17)	Maximum During Short Periods. Number of Minutes. 60. (18)	Mean for month of maximum record. (19)	Maximum. In 24 hours. (20)	Maximum. In one calendar month. (21)	Annual. Minimum. (22)	Annual. Mean. (23)	Annual. Maximum. (24)	(25)
478	Pasuruan, Java	16	....	45	1923	....	....	....	....	23.4	28.9	51.5	90.2	....	....	....	....	19.3	....	45.4	88	100	175	4
479	Batavia, Java	23	....	60	1923	....	....	....	....	32.4	45.9	72.1	96.8	....	....	....	....	18.1	....	44.8	66	100	134	4
480	Medan, East Indies	66	....	45	1923	....	....	....	....	17.2	62.2	81.4	103.7	....	....	....	....	11.1	....	21.1	76	100	127	4
481	Padang, Sumatra	3	....	45	1923	....	....	....	....	36.3	136.4	174.9	205.5	....	....	....	....	11.4	....	20.7	78	100	118	4
482	Kuta-Raja, Sumatra	23	....	45	1923	....	....	....	....	22.7	47.0	65.0	86.5	....	....	....	....	10.4	....	35.0	73	100	133	4
483	Singapore, Malay	....	....	....	....	....	....	....	....	....	32.7	92.0	158.7	....	....	....	....	....	....	....	36	100	173	12
484	Pontianak, Borneo	10	....	45	1923	....	....	....	....	33.9	99.3	126.1	167.1	....	....	....	....	12.0	....	26.8	79	100	132	4
485	Sandakan, Borneo	105	....	33	1920	....	....	....	....	61.1	57.8	119.7	177.6	....	....	....	....	15.4	....	51.0	48	100	148	4
486	Menado, East Indies	13	....	45	1923	....	....	....	....	47.4	62.6	103.7	158.9	....	....	....	....	17.6	....	45.6	60	100	153	4
487	Amboina, East Indies	3	....	45	1923	....	....	....	....	62.2	59.9	134.8	242.7	....	....	....	....	18.3	....	49.1	44	100	180	4
488	Zamboanga, Philippine Islands	10	17	19	1919	....	....	....	6.3	13.1	18.0	39.1	60.0	....	....	....	....	13.0	16.1	33.5	46	100	154	28; 15
489	Iloilo, Philippine Islands	23	17	25	1922	....	....	....	7.9	55.3	45.3	86.3	127.5	....	....	....	....	19.8	9.2	64.0	53	100	148	28; 15
490	Legaspi, Philippine Islands	31	17	20	1922	....	....	....	11.7	44.5	46.6	127.0	202.3	....	....	....	....	15.7	9.8	35.0	38	100	159	28; 4
491	Manila, Philippine Islands	56	37	62	1926	....	....	2.56	13.2	78.1	35.7	80.0	154.4	....	....	....	3.2	20.4	10.5	97.6	45	100	193	4; 28; 13
492	Baguio, Philippine Islands	4 959	17	18	1919	....	....	3.56	{46.0 {34.6	133.0	125.7	182.0	355.8	....	....	....	1.9	22.1	23.3	73.0	69	100	196	28; 13
493	Aparri, N. Philippine Islands	20	17	27	1922	....	....	....	11.3	39.2	46.0	87.2	118.1	....	....	....	....	8.7	12.9	44.9	53	100	135	4; 28
494	Kashoryo, Formosa	1 373	....	12	1918	....	....	....	....	68.7	184.8	282.5	331.0	....	....	....	....	9.8	....	24.3	65	100	117	13; 28
495	Taihoku, Formosa	30	22	24	1920	....	....	....	11.2	37.0	63.0	83.2	110.0	....	....	....	....	14.0	13.6	44.4	76	100	132	4; 13
496	Nagasaki, Japan	433	....	42	1920	....	....	....	....	38.8	40.5	76.9	106.3	....	....	....	....	17.6	....	50.5	53	100	138	4
497	Kioto, Japan	160	....	40	1920	....	....	....	....	24.7	44.5	61.5	81.1	....	....	....	....	12.2	....	40.0	72	100	132	4
498	Tokyo, Japan	70	....	45	1920	....	....	....	....	18.1	45.7	59.8	86.4	....	....	....	....	14.6	....	30.4	76	100	145	4
499	Miyako, Japan	99	....	37	1920	....	....	....	....	22.7	34.6	55.6	80.7	....	....	....	....	12.6	....	41.0	63	100	145	4
500	Nemuro, Japan	89	....	37	1920	....	....	....	....	11.9	25.2	37.8	57.5	....	....	....	....	11.5	....	31.5	67	100	152	4
501	Vladivostok, Siberia	98	....	14	1890	....	....	....	....	13.8	9.4	19.5	33.6	1.4	....	....	....	17.8	....	70.8	48	100	173	20; 21; 12
502	Chemulpo, Korea	222	....	16	1920	....	....	....	....	17.9	25.0	37.4	54.7	....	....	....	....	22.9	....	48.0	67	100	146	4
503	Peking, China	132	31	33	1882	....	....	....	9.9	19.8	9.5	24.6	41.9	1.3	....	....	....	34.2	40.3	80.7	39	100	170	20
504	Zi-Ka-Wei, China	23	....	46	1920	....	....	....	....	19.4	27.9	45.1	62.5	....	....	....	....	16.1	....	43.0	62	100	139	4
505	Hong Kong, China	108	....	{40 {37	1895± 1920	....	....	....	....	.... 48.8	.... 45.8	90.0 84.3	.... 119.7	....	....	....	....	.... 13.8	....	.... 58.0	 54	 100	 142	13 4
506	Saigon, Indo-China	36	....	15	1920	....	....	....	....	23.7	61.9	77.2	107.0	....	....	....	....	14.4	....	30.8	80	100	139	4
507	Mergui, Burma	66	....	56	1920	....	....	....	....	62.7	132.0	162.3	213.0	....	....	....	....	18.6	....	38.6	81	100	131	4
508	Port Blair, Andaman Island	59	....	53	1920	....	....	....	....	41.5	83.9	118.6	150.2	....	....	....	....	18.4	....	38.0	73	100	130	4
509	Moulmein, Burma	77	....	71	1920	....	....	....	....	72.2	184.2	190.2	279.0	....	....	....	....	23.9	....	38.0	65	100	146	4
510	Rangoon, Burma	18	....	61	1920	....	....	....	....	35.7	69.0	99.6	143.4	....	....	....	....	21.6	....	35.5	69	100	144	4
511	Akyab, Burma	20	....	63	1920	....	....	....	....	97.8	142.0	203.8	323.5	....	....	....	....	20.8	....	47.8	70	100	158	4
512	Mandalay, Burma	250	....	35	1920	....	....	....	....	14.2	16.8	35.1	49.7	....	....	....	....	12.4	....	40.4	48	100	142	4
513	Gauhati, N. E. India	196	....	73	1920	....	....	....	....	34.2	46.6	66.5	99.1	....	....	....	....	18.0	....	51.5	70	100	149	4
514	Shillong, N. E. India	4 920	....	55	1920	....	....	....	....	39.9	51.8	83.1	121.2	....	....	....	....	16.0	....	48.0	62	100	146	4
515	Cherrapunji, N. E. India	4 309	....	62	1920	....	....	....	40.0	366.1	282.6	456.9	905+	....	....	....	....	23.5	8.8	80.1	62	100	198	4
516	Calcutta, N. E. India	21	....	92	1920	....	....	4.0	....	45.4	29.4	64.3	93.2	....	....	....	....	15.5	6.3	70.8	61	100	146	4; 8
517	Patna, E. India	183	....	73	1920	....	....	....	....	30.3	21.8	45.9	77.1	....	....	....	....	26.0	....	68.0	48	100	168	4
518	Nagpur, Central India	1 017	....	73	1920	....	....	3.6	....	24.8	14.4	46.4	65.3	....	....	....	7.8	28.7	....	52.8	31	100	141	4; 8
519	Waltair, India	38	....	55	1920	....	....	....	....	30.2	17.1	41.0	79.5	....	....	....	....	22.9	....	73.8	42	100	194	4
520	Masulipatam, India	15	....	58	1920	....	....	....	....	26.6	21.3	39.8	69.5	....	....	....	....	20.7	....	67.0	53	100	175	4
521	Madras, India	22	....	108	1920	....	....	0.94	25.0	42.8	18.4	49.6	88.4	....	....	1.9	....	27.5	50.0	86.2	37	100	178	4; 8
522	Trincomalee, Ceylon	99	....	52	1920	....	....	....	....	35.2	34.9	62.4	95.1	....	....	....	....	22.6	....	96.4	56	100	152	4
523	Nuwara Eliya, Ceylon	6 188	....	52	1920	....	....	....	....	35.0	68.4	92.8	131.5	....	....	....	....	12.8	....	37.6	74	100	142	4
524	Colombo, Ceylon	40	....	51	1920	....	....	....	....	35.3	45.7	80.1	139.7	....	....	....	....	16.6	....	44.0	57	100	174	4
525	Kodiakanal, S. India	7 688	....	21	1920	....	....	....	....	16.8	46.6	62.3	79.7	....	....	....	....	15.5	....	27.0	75	100	128	4
526	Cochin, S. India	9	....	65	1920	....	....	....	....	49.5	80.6	114.5	165.0	....	....	....	....	24.2	....	42.3	70	100	144	4
527	Mangalore, S. India	72	....	66	1920	....	....	....	....	70.4	89.4	126.9	182.3	....	....	....	....	29.0	....	54.7	69	100	142	4
528	Bangalore, S. India	3 021	....	55	1920	....	....	....	....	19.5	16.0	35.3	56.6	....	....	....	....	17.0	....	55.1	45	100	160	4
529	Belgaum, S. India	2 562	....	69	1920	....	....	....	....	42.2	28.7	49.9	73.4	....	....	....	....	31.5	....	84.5	58	100	147	4
530	Bombay, S. India	37	....	104	1920	....	....	3.0	16.0	39.0	33.4	72.4	114.9	....	....	....	4.1	32.5	32.1	81.6	46	100	159	4; 8
531	Ahmadabad, S. India	163	....	55	1920	....	....	....	....	37.5	4.8	28.9	51.2	....	....	....	....	39.2	....	130.0	17	100	177	4
532	Allahabad, S. India	309	....	73	1920	....	....	....	....	30.5	15.7	39.0	76.2	....	....	....	....	26.2	....	78.1	40	100	195	4
533	Jaipur, India	1 431	....	52	1920	....	....	....	....	21.8	4.7	23.9	55.3	....	....	....	....	30.8	....	91.0	20	100	232	4
534	Simla, India	7 232	....	69	1920	....	....	....	....	42.5	40.4	80.0	109.7	....	....	....	....	26.0	....	53.2	50	100	137	4
535	Leh, India	11 503	....	45	1920	....	....	....	....	4.0	0.4	3.2	9.1	....	....	....	....	5.0	....	125.0	12	100	284	4
536	Lahore, India	702	....	63	1920	0.90	2.00	4.50	9.0	20.4	6.2	20.1	37.8	....	....	....	....	25.0	....	101.5	31	100	188	4
537	Peshawar, N. W. India	1 113	....	62	1920	....	....	....	....	17.7	4.1	13.5	27.9	....	....	....	....	16.1	....	151.0	30	100	206	4
538	Quetta, Baluchistan	5 502	....	43	1920	....	....	....	....	7.5	3.9	9.3	21.6	....	....	....	....	19.5	....	80.8	42	100	233	4
539	Khelat, Baluchistan	6 630	....	41	1920	....	....	....	....	7.2	4.1	7.4	13.6	....	....	....	....	18.9	....	97.1	55	100	184	4
540	Hyderabad, N. W. India	98	....	55	1920	....	....	....	6.9	15.8	1.0	7.6	21.1	....	....	....	....	37.0	91.0	207.9	13	100	277	4; 8
541	Karachi, N. W. India	13	....	65	1920	....	....	....	....	18.6	0.5	7.4	28.0	....	....	....	....	38.3	....	251.3	7	100	378	4
542	Muscat, E. Arabia	20	....	27	1920	....	....	....	....	4.5	0.9	4.2	10.4	5.5	....	....	....	27.6	....	107.1	21	100	247	4
543	Jask, Persia	13	....	28	1920	....	....	....	....	4.8	0.2	4.5	7.7	....	....	....	....	17.8	....	106.7	4	100	182	4
544	Bushire, Persia	14	....	44	1920	....	....	....	....	12.9	0.0	10.4	26.6	....	....	....	....	25.7	....	124.0	0	100	256	4
545	Meshed, Persia	3 104	....	28	1920	....	....	....	....	5.2	3.1	9.2	16.8	....	....	....	....	24.4	....	56.8	34	100	182	4
546	Teheran, Persia	4 002	....	27	1920	....	....	....	....	7.5	3.6	9.5	15.8	....	....	....	....	17.4	....	78.9	38	100	161	4
547	Baghdad, Mesopotamia	125	....	31	1920	....	....	....	2.5	7.9	1.3	7.1	22.3	3.7	....	....	....	19.6	37.2	111.1	18	100	314	4
548	Aden, S. Arabia	34	....	40	1920	....	....	....	....	6.6	0.0	1.8	8.8	....	....	....	....	20.6	....	366.7	0	100	478	4
549	Khartoum, Sudan	1 280	22	24	1922	....	....	....	3.5	7.4	1.3	5.7	14.4	....	....	....	....	45.6	61.5	130.0	23	100	253	4; 19
550	Deim Zubeir, Sudan	2 168	14	15	1920	....	....	....	5.4	23.9	......	45.7	78.5	....	....	....	....	17.0	11.8	52.3	....	100	172	19
551	Gore, Abyssinia	6 890	13	13	1920	....	....	....	3.5	17.0	......	75.7	92.6	....	....	....	....	15.0	4.6	22.5	....	100	122	19
552	Adis-Ababa, Abyssinia	8 006	13	23	1920	....	....	....	3.0	22.5	......	49.6	74.9	....	....	....	....	15.2	6.0	45.4	....	100	151	19
553	Entebbe, Uganda	3 847	....	25	1920	....	....	....	....	15.8	47.2	58.0	75.7	....	....	....	....	16.8	....	27.3	81	100	131	4
554	Dar-es-Salaam, E. Africa	249	....	20	1912	....	....	....	....	23.8	19.4	42.3	56.9	....	....	....	....	28.0	....	56.2	46	100	134	4
555	Zanzibar, E. Africa	56	....	30	1920	....	....	....	....	3.3	27.5	60.2	92.9	....	....	....	....	23.4	....	55.8	46	100	154	4
556	Seychelles Is., E. Africa	.....	....	30	1920	....	....	....	....	31.4	66.5	94.5	132.8	....	....	....	....	17.9	....	33.3	70	100	140	4
557	Tananarive, Madagascar	4 600	....	34	1923	....	....	....	....	26.4	27.2	53.7	69.6	....	....	....	....	21.4	....	49.1	51	100	130	4
558	Salisbury, Rhodesia	4 860	....	28	1923	....	....	....	....	18.8	21.7	31.9	42.6	....	....	....	....	23.2	....	59.0	68	100	134	4
559	Bulawayo, Rhodesia	4 443	....	27	1923	....	....	....	....	13.4	13.1	23.6	36.8	....	....	....	....	21.9	....	56.8	51	100	156	4
560	Belvedere, S. Africa	5 000	....	17	1925	....	....	....	....	....	......	51.4	....	....	....	....	....	....	....	....	....	100	....	21
561	Johannesburg, S. Africa	5 925	....	37	1924	....	....	....	....	20.0	19.8	33.2	54.9	1.1	....	....	....	18.6	....	60.2	60	100	165	4
562	Kroonstad, S. Africa	4 300	....	45	1925	....	....	....	....	....	......	24.2	....	....	....	....	....	....	....	....	....	100	....	21
563	Aliwal, S. Africa	4 352	....	42	1924	....	....	....	....	12.2	8.9	20.4	35.8	....	....	....	....	14.3	....	59.8	44	100	176	4
564	Kokstad, S. Africa	4 280	....	40	1925	....	....	....	....	....	......	27.4	....	....	....	....	....	....	....	....	....	100	....	21
565	Durban, S. Africa	265	....	39	1920	....	....	....	....	26.7	27.2	42.7	85.5	0.8	....	....	....	12.0	....	62.6	64	100	200	4
566	Port Elizabeth, S. Africa	181	....	59	1924	....	....	....	....	9.3	10.4	22.5	34.1	1.3	....	....	....	9.3	....	41.2	46	100	152	4
567	Cape Town, S. Africa	40	....	87	1924	....	....	....	....	13.5	17.5	25.3	41.0	....	....	....	....	15.1	....	53.3	69	100	162	4
568	Carnarvon, S. Africa	4 060	....	46	1925	....	....	....	....	....	......	8.1	....	....	....	....	....	....	....	....	....	100	....	21
569	Springbok, S. Africa	3 200	....	48	1925	....	....	....	....	....	......	9.6	....	....	....	....	....	....	....	....	....	100	....	21
570	O'Okiep, S. Africa	3 035	....	42	1924	....	....	....	....	4.3	2.1	6.7	11.7	....	....	....	....	16.8	....	64.0	31	100	174	4
571	Port Nolloth, S. Africa	25	....	40	1925	....	....	....	....	....	......	2.5	....	....	....	....	....	....	....	....	....	100	....	21
572	Bethany, S. W. Africa	.....	....	14	1913	....	....	....	....	4.1	1.2	4.0	9.1	....	....	....	....	23.7	....	102.5	30	100	227	15
573	Hasuur, S. W. Africa	.....	....	14	1913	....	....	....	....	7.1	0.3	7.6	25.6	....	....	....	....	29.6	....	93.0	4	100	338	15
574	Steinkopf, S. W. Africa	3 000	....	36	1925	....	....	....	....	....	......	5.6	....	....	....	....	....	....	....	....	....	100	....	21
575	Pella, S. W. Africa	1 500	....	47	1925	....	....	....	....	....	......	3.3	....	....	....	....	....	....	....	....	....	100	....	21
576	Rietfontein, S. W. Africa	2 720	....	17	1925	....	....	....	....	....	......	7.2	....	4.3	....	....	....	....	....	....	....	100	....	21
577	Kenhardt, S. Africa	2 704	....	45	1925	....	....	....	....	....	......	5.8	....	....	....	....	....	....	....	....	....	100	....	21
578	Prieska, S. Africa	3 100	....	48	1925	....	....	....	....	....	......	9.6	....	....	....	....	....	....	....	....	....	100	....	21
579	Hopetown, S. Africa	3 604	....	46	1925	....	....	....	....	....	......	12.6	....	....	....	....	....	....	....	....	....	100	....	21
580	Kimberley, S. Africa	3 947	....	50	1923	....	....	....	....	9.0	8.5	16.9	31.3	1.6	....	....	....	17.4	....	53.2	50	100	186	4
581	Vryburg, S. Africa	3 890	....	33	1925	....	....	....	....	....	......	19.2	....	....	....	....	....	....	....	....	....	100	....	21
582	Mafeking, S. Africa	4 194	....	23	1925	....	....	....	....	....	......	22.2	....	1.8	....	....	....	....	....	....	....	100	....	21
583	Palapye Road, S. Africa	.....	....	8	......	....	....	....	....	....	......	16.5	....	....	....	....	....	....	....	....	....	100	....	15
584	Tsau, S. W. Africa	.....	....	4	......	....	....	....	....	....	......	12.6	....	....	....	....	....	....	....	....	....	100	....	15
585	Windhoek, S. W. Africa	5 461	....	22	1913	....	....	....	....	12.2	7.3	15.2	29.8	....	....	....	....	25.4	....	80.1	48	100	196	15
586	Swakopmund, S. W. Africa	26	....	15	1913	....	....	....	....	0.8	0.1	0.6	1.2	....	....	....	....	16.7	....	133.3	17	100	200	15
587	Waterberg, S. W. Africa	5 248	....	21	1913	....	....	....	....	20.3	7.3	21.1	41.6	....	....	....	....	26.3	....	96.0	38	100	197	15
588	Mossamedes, Angola	.....	....	10	......	....	....	....	....	....	......	1.47	....	....	....	....	....	....	....	....	....	100	....	15
589	Loanda, Angola	.....	....	42	......	....	....	....	....	....	......	12.4	....	....	....	....	....	....	....	....	....	100	....	15
590	Cazengo, Angola	.....	....	10	......	....	....	....	....	....	......	42.2	....	....	....	....	....	....	....	....	....	100	....	15
591	Camaxilio, Angola	.....	....	4	......	....	....	....	....	....	......	52.6	....	....	....	....	....	....	....	....	....	100	....	15

(Continued.)

Station No.	January.			February.			March.			April.			May.			June.			July.			August.			September.			October.			November.			December.		
	Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.	
		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
478	9.92	...	23.4	10.83	...	18.9	8.23	...	17.5	5.16	...	18.1	3.54	...	19.3	2.32	...	10.1	1.10	...	10.4	0.24	...	4.1	0.28	...	4.1	0.83	...	8.1	2.48	...	12.1	6.54	...	11.1
479	13.03	...	32.4	12.83	...	27.8	7.80	...	19.4	5.16	...	14.7	3.98	...	13.1	3.74	...	9.6	2.60	...	8.3	1.69	...	7.5	2.87	...	9.6	4.49	...	11.7	5.47	...	11.0	8.46	...	22.4
480	5.55	...	12.4	3.66	...	10.6	4.17	...	13.8	5.20	...	11.5	7.24	...	15.4	5.12	...	16.5	5.35	...	13.9	7.24	...	14.9	8.58	...	13.4	10.20	...	14.2	10.00	...	16.9	9.09	...	17.2
481	13.58	...	25.6	10.24	...	22.7	11.61	...	21.6	14.49	...	28.5	12.24	...	23.8	12.76	...	22.6	11.22	...	27.0	12.58	...	27.9	16.02	...	29.5	19.92	...	36.3	20.12	...	22.9	19.09	...	33.3
482	6.91	...	13.6	3.50	...	9.5	3.50	...	8.8	4.21	...	14.0	6.06	...	18.5	3.62	...	9.0	4.25	...	16.5	4.45	...	15.0	6.77	...	22.7	7.20	...	15.9	7.24	...	19.3	8.27	...	20.7
483	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....
484	10.73	...	18.9	7.80	...	20.8	9.76	...	23.0	10.87	...	21.1	10.59	...	20.4	8.74	...	17.8	6.50	...	19.2	8.66	...	18.4	8.27	...	20.0	15.08	...	33.9	15.91	...	24.8	13.19	...	30.7
485	19.45	...	61.1	9.62	...	34.5	8.05	...	24.9	4.07	...	9.4	5.91	...	12.4	7.30	...	17.1	6.55	...	11.9	8.06	...	29.2	9.39	...	20.4	10.00	...	16.7	14.66	...	31.4	17.66	...	32.5
486	18.27	...	47.4	14.13	...	26.7	11.10	...	26.9	7.82	...	19.4	6.46	...	14.8	6.42	...	11.3	4.80	...	14.6	3.70	...	9.6	3.35	...	9.6	4.72	...	15.6	8.50	...	29.5	14.45	...	45.0
487	5.12	...	11.6	4.61	...	10.5	5.23	...	10.2	11.30	...	24.9	19.84	...	55.4	24.60	...	62.2	23.07	...	56.3	15.94	...	57.1	9.09	...	38.2	6.18	...	12.9	4.37	...	10.8	5.39	...	11.6
488	2.36	5.0	11.9	2.01	2.6	7.2	1.40	2.9	4.1	1.60	2.0	4.3	2.92	2.1	6.4	4.01	1.6	7.1	4.12	2.3	8.4	3.45	2.2	7.6	4.20	4.5	11.2	5.06	4.8	13.1	3.89	2.1	9.4	4.15	6.3	11.0
489	2.00	2.5	7.8	1.48	2.2	8.2	1.20	3.1	4.6	1.46	2.8	6.2	6.66	5.4	19.2	9.59	5.2	18.0	17.02	7.9	55.3	13.06	5.2	30.0	12.33	4.1	24.3	10.09	7.2	22.0	6.92	6.5	18.1	4.59	3.1	30.8
490	14.63	9.4	31.4	10.87	7.2	29.0	7.39	4.1	27.1	4.57	4.1	15.2	5.04	5.0	18.8	8.04	6.8	20.0	9.93	5.1	19.8	7.29	3.9	14.1	9.86	7.7	20.0	12.80	9.0	24.4	16.43	11.7	41.4	19.92	7.0	44.5
491	0.97	7.2	7.7	0.43	1.7	1.8	0.73	2.4	3.9	1.24	5.6	6.8	4.46	8.3	18.8	9.98	9.9	26.0	17.04	11.5	46.9	16.34	10.9	78.1	14.13	13.2	57.9	7.66	7.6	23.2	5.52	11.0	23.4	2.43	3.5	13.7
492	1.21	3.6	5.8	0.67	1.3	2.1	1.92	2.2	5.9	4.55	5.1	9.7	16.08	16.4	55.0	15.80	19.6	38.8	40.30	{46.0 34.6}	33.0	41.50	23.0	99.6	31.80	21.5	88.3	16.00	27.1	59.5	3.30	4.9	9.3	2.09	7.2	10.9
493	7.57	2.7	39.2	3.83	2.8	19.1	2.45	2.1	7.5	1.78	3.9	8.2	4.41	5.2	13.3	5.32	6.6	13.8	7.14	4.1	19.9	9.41	7.6	19.7	11.30	6.0	23.4	12.05	9.3	30.6	12.30	11.3	34.8	12.66	7.1	25.3
494	31.45	...	48.0	23.60	...	37.1	24.39	...	35.3	12.14	...	16.6	17.20	...	25.6	12.92	...	21.6	11.60	...	24.2	12.24	...	24.5	27.70	...	68.7	37.65	...	59.6	34.30	...	59.5	37.62	...	47.5
495	3.46	2.0	8.5	5.04	2.2	9.9	6.91	2.6	15.7	5.46	4.4	11.2	9.26	6.2	20.1	10.8	7.8	27.1	8.85	7.5	18.6	11.70	11.3	37.0	10.30	8.2	30.8	8.08	6.5	25.9	3.09	2.3	6.7	3.25	3.1	10.3
496	3.10	...	7.7	3.47	...	7.6	5.21	...	9.2	8.06	...	20.1	7.35	...	15.8	13.56	...	38.8	9.27	...	31.4	7.24	...	19.0	8.57	...	22.6	4.61	...	12.9	3.36	...	8.5	3.28	...	8.6
497	2.46	...	6.4	2.54	...	6.6	4.44	...	7.6	6.26	...	12.8	5.57	...	11.3	9.26	...	24.3	7.56	...	24.7	5.70	...	13.7	7.94	...	17.7	5.25	...	10.1	3.01	...	6.9	2.15	...	4.7
498	2.20	...	9.0	2.81	...	6.6	4.40	...	8.1	4.92	...	8.3	5.67	...	11.3	6.48	...	15.0	5.30	...	12.6	5.74	...	16.5	8.70	...	18.1	7.38	...	14.0	4.23	...	14.9	2.11	...	5.7
499	2.72	...	8.5	2.65	...	7.4	3.45	...	7.7	3.86	...	8.2	4.69	...	16.9	4.97	...	14.5	5.30	...	16.1	7.02	...	22.7	8.49	...	16.3	6.68	...	19.4	3.22	...	10.6	2.48	...	9.1
500	1.48	...	3.7	1.00	...	4.1	2.19	...	7.9	2.87	...	5.4	3.71	...	7.9	3.06	...	8.5	3.79	...	8.2	4.34	...	11.9	5.54	...	10.6	3.62	...	8.4	3.25	...	9.2	2.30	...	7.4
501	0.08	...	0.4	0.16	...	0.4	0.28	...	0.5	1.13	...	2.9	1.25	...	3.2	1.47	...	1.9	2.18	...	1.9	3.48	...	13.8	2.36	...	4.4	1.61	...	4.8	0.50	...	2.4	0.19	...	0.6
502	0.79	...	2.3	0.54	...	2.4	1.05	...	6.8	2.57	...	6.1	3.20	...	5.8	4.26	...	11.5	8.57	...	17.9	8.16	...	15.9	4.26	...	17.3	1.57	...	3.5	1.70	...	3.0	0.74	...	3.1
503	0.12	0.9	1.2	0.19	0.5	1.1	0.25	0.7	1.1	0.64	1.3	2.2	1.53	2.4	5.3	3.36	4.6	8.1	8.40	9.9	19.8	6.47	5.7	14.6	2.65	4.1	11.2	0.61	1.5	1.8	0.28	0.9	0.9	0.07	0.8	0.8
504	2.02	...	7.8	2.29	...	7.1	3.50	...	6.3	3.71	...	9.4	3.51	...	7.2	7.27	...	19.4	6.02	...	12.0	5.87	...	13.8	4.45	...	10.8	3.22	...	12.0	2.07	...	5.9	1.39	...	6.0
505	{0.98	...	....	1.30	...	....	3.27	...	....	5.43	...	....	12.45	...	....	16.34	...	....	15.83	...	....	14.80	...	....	12.52	...	....	5.25	...	....	1.14	...	....	0.98}	...	4.9
	{1.33	...	8.4	1.63	...	7.9	2.72	...	11.5	5.36	...	14.9	11.65	...	48.8	15.93	...	34.4	13.83	...	30.1	14.12	...	30.1	9.84	...	30.6	4.88	...	24.0	1.83	...	8.8	1.14}		
506	0.90	...	4.3	0.13	...	0.4	0.28	...	1.3	1.76	...	7.8	8.35	...	11.8	12.61	...	18.6	11.11	...	23.4	10.96	...	19.7	13.39	...	18.3	11.07	...	23.7	3.07	...	11.3	2.15	...	6.8
507	0.79	...	5.2	1.91	...	6.6	2.80	...	10.4	5.30	...	23.1	17.35	...	36.4	20.24	...	62.7	31.48	...	53.5	29.54	...	49.6	26.35	...	43.8	12.42	...	25.9	3.48	...	11.9	0.65	...	6.9
508	1.48	...	23.0	1.12	...	8.6	0.65	...	8.1	2.42	...	13.3	15.92	...	40.6	18.91	...	41.5	15.32	...	31.2	15.06	...	34.3	18.07	...	22.3	11.41	...	20.3	9.12	...	25.5	6.17	...	17.6
509	0.15	...	7.8	0.10	...	2.3	0.36	...	3.7	2.94	...	10.5	20.63	...	45.8	37.85	...	64.9	45.10	...	68.3	43.75	...	72.2	29.06	...	63.3	8.49	...	21.0	1.72	...	10.7	0.17	...	3.8
510	0.18	...	5.5	0.20	...	3.4	0.28	...	3.7	1.44	...	9.6	12.13	...	27.2	18.39	...	32.4	21.52	...	35.7	19.67	...	30.8	15.41	...	25.9	7.20	...	16.3	2.80	...	16.9	0.31	...	4.5
511	0.11	...	1.5	0.18	...	3.6	0.51	...	4.9	2.01	...	15.8	13.70	...	62.1	49.42	...	97.6	53.68	...	97.8	42.47	...	75.9	24.58	...	54.2	11.56	...	27.6	4.98	...	35.2	0.57	...	7.4
512	0.08	...	0.6	0.08	...	0.6	0.19	...	1.6	1.12	...	4.4	3.78	...	11.1	5.52	...	13.6	3.29	...	9.2	4.58	...	10.8	5.76	...	14.1	4.72	...	14.3	1.68	...	9.1	[illegible]	...	1.9
513	0.52	...	2.3	0.96	...	6.2	2.46	...	8.6	6.40	...	13.4	9.51	...	24.1	12.62	...	34.2	12.25	...	28.6	10.66	...	19.8	7.44	...	18.3	2.91	...	13.9	0.54	...	5.0	0.22	...	1.9
514	0.43	...	2.2	0.91	...	3.1	1.88	...	5.1	4.68	...	11.8	10.15	...	24.6	16.53	...	35.9	13.68	...	36.3	13.24	...	30.6	13.37	...	39.9	6.55	...	20.3	1.26	...	7.1	0.22	...	1.6
515	0.67	...	4.3	2.31	...	18.6	10.57	...	54.9	31.28	...	93.3	50.85	...	141.8	108.6	...	200.1	107.4	...	266.1	81.47	...	149.3	49.41	...	125.1	16.78	...	58.1	2.27	...	13.4	0.38	...	3.2
516	0.42	...	2.9	0.99	...	8.0	1.38	...	6.3	2.18	...	7.3	5.56	...	17.1	11.91	...	31.1	12.70	...	24.8	13.38	...	26.9	10.01	...	45.4	4.90	...	16.2	0.65	...	5.6	0.24	...	3.0
517	0.61	...	3.7	0.63	...	2.8	0.41	...	2.8	0.31	...	2.0	1.61	...	9.6	7.73	...	28.9	11.30	...	23.4	16.91	...	30.2	8.41	...	26.2	3.59	...	14.6	0.25	...	2.8	0.11	...	1.8
518	0.84	...	4.8	0.54	...	4.6	0.57	...	3.8	0.53	...	3.9	0.78	...	5.9	8.79	...	22.2	13.34	...	24.8	10.34	...	22.6	7.87	...	24.7	2.06	...	10.5	0.86	...	4.8	0.48	...	8.7
519	0.36	...	6.5	0.48	...	4.8	0.32	...	2.9	0.71	...	7.6	2.31	...	14.5	4.56	...	14.2	4.66	...	10.8	5.75	...	15.6	7.21	...	22.0	9.36	...	30.2	4.10	...	13.6	1.21	...	25.8
520	0.22	...	3.2	0.34	...	6.9	0.32	...	4.4	0.46	...	4.5	1.39	...	9.8	4.32	...	11.7	6.16	...	18.7	6.46	...	19.9	6.53	...	17.4	8.21	...	26.6	4.70	...	20.7	0.69	...	7.2
521	1.14	...	9.6	0.30	...	6.3	0.34	...	6.7	0.63	...	7.5	1.84	...	23.3	1.97	...	8.6	3.84	...	11.7	4.54	...	13.2	4.86	...	14.9	11.15	...	87.7	13.61	...	42.8	3.35	...	22.8
522	6.08	...	29.1	2.12	...	8.1	1.55	...	8.2	2.03	...	9.4	2.49	...	9.2	1.36	...	6.5	2.08	...	7.8	4.20	...	11.7	4.68	...	11.4	7.85	...	20.7	14.14	...	35.2	12.79	...	31.5
523	5.37	...	24.7	2.09	...	10.8	2.36	...	9.6	5.60	...	13.5	6.81	...	22.1	12.80	...	27.8	11.90	...	35.0	7.82	...	20.8	8.06	...	24.5	10.97	...	28.5	9.09	...	16.8	8.72	...	23.1
524	3.22	...	12.6	1.94	...	5.3	4.28	...	10.5	9.73	...	28.8	10.94	...	22.8	7.32	...	20.0	4.43	...	29.0	3.24	...	19.5	4.76	...	25.1	13.36	...	35.3	11.76	...	28.8	5.12	...	15.5
525	2.88	...	9.9	1.41	...	6.5	3.03	...	5.3	4.25	...	10.0	6.02	...	11.3	4.06	...	8.6	5.02	...	11.5	6.99	...	16.0	7.25	...	13.6	9.68	...	16.8	8.16	...	15.5	4.42	...	12.1
526	0.82	...	5.2	0.78	...	5.8	1.75	...	17.9	3.68	...	14.7	11.40	...	27.1	27.79	...	49.5	25.27	...	44.3	12.54	...	29.4	9.21	...	22.5	12.96	...	27.1	6.70	...	16.5	1.91	...	7.2
527	0.11	...	4.2	0.08	...	1.8	0.10	...	1.6	1.72	...	11.7	6.88	...	44.4	38.09	...	69.9	37.40	...	70.4	22.88	...	43.9	10.74	...	29.5	7.87	...	17.8	2.31	...	9.8	0.51	...	5.9
528	0.26	...	4.0	0.15	...	3.1	0.54	...	6.4	1.34	...	5.0	4.66	...	15.5	3.09	...	8.6	4.02	...	11.4	5.61	...	14.2	6.83	...	19.3	5.99	...	19.8	2.26	...	9.9	0.58	...	5.8
529	0.11	...	2.3	0.04	...	0.9	0.37	...	3.0	1.83	...	6.7	2.66	...	10.6	8.85	...	18.6	15.75	...	42.2	9.42	...	27.0	4.37	...	12.4	4.85	...	11.8	1.45	...	10.7	0.36	...	7.6
530	0.12	...	8.2	0.06	...	1.7	0.04	...	1.5	0.04	...	1.6	0.68	...	11.0	19.86	16.0	49.1	24.16	...	59.0	14.55	...	37.4	10.60	...	31.8	1.66	...	20.0	0.43	...	5.8	0.05	...	1.1
531	0.02	...	0.4	0.10	...	1.0	0.06	...	0.6	0.03	...	0.5	0.51	...	4.2	3.88	...	17.1	11.32	...	37.5	8.29	...	34.7	4.00	...	20.1	0.58	...	7.1	0.14	...	2.4	0.08	...	0.5
532	0.75	...	5.0	0.55	...	3.1	0.38	...	3.5	0.15	...	1.4	0.31	...	2.5	4.68	...	20.7	12.01	...	25.7	10.99	...	30.5	6.32	...	18.7	2.33	...	22.5	0.29	...	5.3	0.21	...	2.9
533	0.41	...	2.6	0.28	...	2.3	0.35	...	1.8	0.20	...	2.5	0.61	...	2.9	2.59	...	13.6	8.28	...	21.5	7.30	...	21.8	3.19	...	14.7	0.30	...	2.8	0.13	...	2.4	0.30	...	1.5
534	3.60	...	10.0	3.73	...	9.0	3.32	...	9.1	2.73	...	14.7	2.92	...	13.3	8.76	...	18.6	21.09	...	36.4	20.74	...	42.5	7.48	...	16.9	1.42	...	9.6	0.50	...	5.0	1.34	...	8.1
535	0.36	...	1.6	0.31	...	1.7	0.27	...	1.5	0.24	...	1.3	0.23	...	2.2	0.19	...	1.1	0.47	...	1.9	0.52	...	2.0	0.26	...	2.7	0.16	...	4.0	0.02	...	0.2	0.16	...	1.1
536	0.90	...	3.9	1.01	...	4.7	0.96	...	6.5	0.56	...	3.3	0.72	...	4.4	1.68	...	8.8	6.11	...	17.3	5.03	...	20.4	2.25	...	11.4	0.89	...	3.6	0.10	...	1.3	0.43	...	2.6
537	1.82	...	5.1	1.37	...	8.0	2.10	...	7.5	1.82	...	7.3	0.72	...	5.2	0.32	...	3.8	1.41	...	6.9	2.17	...	17.7	0.81	...	7.0	0.20	...	1.6	0.42	...	8.5	0.62	...	6.3
538	1.88	...	6.4	1.81	...	7.5	1.88	...	5.3	1.01	...	5.1	0.37	...	2.0	0.15	...	1.1	0.36	...	2.7	0.42	...	2.5	0.07	...	1.1	0.13	...	1.9	0.32	...	3.7	0.33	...	4.2
539	1.47	...	4.7	1.40	...	7.2	1.26	...	3.4	0.57	...	2.5	0.24	...	1.3	0.16	...	2.5	0.56	...	2.7	0.41	...	2.4	0.05	...	0.8	0.10	...	2.4	0.34	...	2.1	0.85	...	6.3
540	0.22	...	1.9	0.26	...	2.2	0.21	...	3.6	0.12	...	3.4	0.15	...	2.2	0.47	...	3.6	2.81	6.9	15.8	2.62	...	11.8	0.62	...	5.7	0.01	...	0.4	0.07	...	1.9	0.05	...	0.6
541	0.56	...	3.4	0.36	...	2.9	0.26	...	3.8	0.12	...	4.7	0.07	...	1.8	0.64	...	10.6	2.82	...	18.6	1.68	...	14.1	0.56	...	8.3	0.02	...	1.6	0.12	...	4.7	0.17	...	1.4
542	1.16	...	4.5	0.84	...	3.9	0.50	...	2.5	0.40	...	3.9	0.01	...	0.2	0.11	...	2.5	0.02	...	0.3	0.03	...	0.6	0.00	...	0.0	0.09	...	1.0	0.33	...	3.0	0.59	...	2.5
543	1.12	...	4.5	0.88	...	3.1	0.80	...	4.8	0.18	...	2.3	0.01	...	0.4	0.06	...	0.8	0.006	...	0.1	0.01	...	0.3	0.00	...	0.0	0.05	...	0.6	0.27	...	2.0	1.12	...	4.7
544	2.67	...	12.9	1.88	...	6.7	0.95	...	4.7	0.49	...	2.1	0.02	...	0.2	0.00	...	0.0	0.00	...	0.0	0.005	...	0.2	0.00	...	0.0	0.06	...	1.0	1.36	...	11.7	2.94	...	12.7
545	0.82	...	1.9	1.00	...	3.3	2.24	...	5.2	1.84	...	3.9	1.23	...	4.0	0.33	...	2.1	0.06	...	1.3	0.03	...	0.3	0.02	...	0.1	0.39	...	1.8	0.00	...	2.3	0.66	...	2.3
546	1.65	...	7.5	1.14	...	3.3	1.96	...	6.1	1.27	...	4.0	0.60	...	2.4	0.08	...	0.6	0.16	...	1.8	0.04	...	0.3	0.06	...	0.7	0.34	...	1.4	0.88	...	2.6	1.22	...	2.6
547	1.17	...	4.8	1.32	...	7.9	1.34	...	8.1	0.93	...	3.2	0.23	...	1.2	0.00	...	0.0	0.00	...	0.0	0.03	...	1.1	0.01	...	1.2	0.08	...	0.5	0.74	...	4.8	1.23	...	4.5
548	0.32	...	1.8	0.19	...	1.6	0.48	...	6.6	0.18	...	2.1	0.12	...	1.4	0.06	...	1.3	0.03	...	0.6	0.12	...	2.0	0.15	...	1.4	0.09	...	2.2	0.08	...	1.3	0.11	...	1.1
549	0.08	Tr.	Tr.	0.00	Tr.	Tr.	0.00	Tr.	Tr.	0.00	Tr.	Tr.	0.12	0.8	0.9	0.31	1.3	1.4	1.76	3.5	5.9	2.60	2.2	7.4	0.70	2.0	1.9	0.20	0.5	2.0	0.00	0.0	0.0	0.00	0.0	0.0
550	0.04	0.7	0.7	0.20	1.0	1.0	0.94	1.7	2.4	2.52	2.1	5.4	4.90	3.9	10.2	6.50	2.2	9.8	9.13	4.9	16.9	7.80	5.4	15.1	7.76	3.2	23.9	5.25	4.6	13.1	0.63	1.0	1.8	0.00	0.0	0.0
551	1.08	1.0	2.2	1.77	1.3	5.2	2.76	1.6	8.7	4.50	2.1	10.3	9.25	1.9	13.7	10.12	2.0	13.7	10.83	2.9	16.8	11.57	3.5	15.3	11.34	2.0	17.0	7.58	2.2	14.4	3.59	2.0	7.7	1.42	1.3	3.5
552	0.89	1.2	2.5	1.90	1.5	6.1	2.76	1.5	7.8	3.43	1.7	5.5	2.95	2.1	10.6	5.75	1.9	11.6	10.98	1.6	15.0	12.09	3.0	16.5	7.55	3.0	22.5	0.79	1.5	3.9	0.55	0.7	3.3	0.34	1.1	1.3
553	2.61	...	8.0	3.60	...	8.3	5.81	...	9.9	9.75	...	15.8	8.51	...	15.8	5.11	...	10.3	2.94	...	7.7	3.06	...	6.7	3.07	...	5.6	3.47	...	6.5	5.01	...	12.0	5.09	...	12.5
554	3.27	...	10.2	2.13	...	6.0	4.83	...	10.5	11.84	...	23.8	7.38	...	14.8	1.10	...	4.6	1.66	...	3.4	1.12	...	4.2	1.15	...	2.6	1.24	...	4.7	2.86	...	10.6	3.74	...	9.7
555	2.83	...	6.8	2.20	...	10.7	6.06	...	10.6	14.07	...	33.3	10.71	...	21.2	1.99	...	8.1	2.36	...	5.1	1.65	...	4.5	2.11	...	6.6	3.63	...	6.8	7.48	...	17.2	5.36	...	17.8
556	14.90	...	31.4	12.48	...	24.9	9.05	...	17.4	6.98	...	17.1	6.02	...	18.2	4.47	...	20.1	2.58	...	6.0	2.25	...	7.1	5.75	...	22.6	5.02	...	13.2	9.44	...	26.2	13.56	...	28.7
557	13.01	...	24.3	11.48	...	26.4	7.41	...	15.2	2.02	...	5.2	0.57	...	1.6	0.27	...	1.0	0.25	...	0.7	0.31	...	1.3	0.57	...	2.5	2.50	...	11.1	5.07	...	15.4	11.25	...	21.6
558	7.47	...	15.7	7.40	...	18.8	4.50	...	11.4	0.99	...	2.9	0.54	...	4.9	0.05	...	0.7	0.02	...	0.4	0.08	...	0.2	0.26	...	3.1	1.16	...	4.2	3.70	...	8.4	5.78	...	12.7
559	5.93	...	12.7	4.01	...	11.6	3.18	...	8.2	0.65	...	3.2	0.28	...	1.3	0.03	...	0.7	0.04	...	0.9	0.03	...	0.5	0.12	...	1.3	0.91	...	3.6	3.31	...	7.7	5.18	...	13.4
560	16.48	...	....	14.71	...	....	13.69	...	....	4.02	...	....	2.60	...	....	0.62	...	....	0.89	...	....	1.72	...	....	1.64	...	....	4.94	...	....	8.48	...	....	11.62	...	....
561	6.17	...	20.0	5.22	...	16.0	4.44	...	10.9	1.74	...	5.2	0.76	...	4.1	0.14	...	0.8	0.33	...	3.4	0.51	...	4.3	0.99	...	2.8	2.56	...	5.8	4.96	...	19.2	6.43	...	10.6
562	4.07	...	....	3.42	...	....	3.56	...	....	1.74	...	....	0.96	...	....	0.29	...	....	0.27	...	....	0.51	...	....	0.88	...	....	1.99	...	....	3.22	...	....	3.30	...	....
563	2.91	...	12.2	3.26	...	8.4	3.28	...	10.1	1.64	...	4.2	0.90	...	4.1	0.58	...	3.4	0.41	...	2.7	0.72	...	3.1	0.85	...	4.4	2.41	...	4.0	1.93	...	6.2	2.46	...	9.2
564	4.44	...	....	4.23	...	....	3.72	...	....	1.54	...	....	1.05	...	....	0.47	...	....	0.35	...	....	0.64	...	....	1.27	...	....	1.90	...	....	3.51	...	....	4.18	...	....
565	4.57	...	9.5	4.91	...	11.1	5.37	...	21.1	3.30	...	15.4	1.90	...	5.8	1.16	...	11.1	1.24	...	15.2	1.74	...	7.9	3.18	...	13.8	5.14	...	26.7	4.97	...	16.6	5.10	...	13.8
566	1.21	...	4.9	1.33	...	5.5	1.79	...	5.3	1.97	...	5.5	2.38	...	8.0	1.66	...	4.8	1.89	...	6.2	2.09	...	5.0	2.24	...	7.7	2.14	...	6.0	2.09	...	9.3	1.70	...	6.2
567	0.70	...	5.1	0.58	...	3.1	0.91	...	4.4	1.92	...	5.9	3.83	...	13.5	4.51	...	13.3	3.65	...	8.6	3.39	...	8.8	2.27	...	6.0	1.64	...	4.7	1.08	...	3.7	0.82	...	3.2
568	0.92	...	....	1.87	...	....	1.60	...	....	0.92	...	....	0.64	...	....	0.35	...	....	0.23	...	....	0.28	...	....	0.30	...	....	0.40	...	....	0.58	...	....	0.47	...	....
569	0.19	...	....	0.83	...	....	0.57	...	....	0.74	...	....	1.33	...	....	1.85	...	....	1.39	...	....	1.32	...	....	0.81	...	....	0.56	...	....	0.28	...	....	0.22	...	....
570	0.19	...	6.8	0.32	...	2.5	0.43	...	1.7	0.64	...	2.4	0.96	...	2.7	1.13	...	4.3	0.77	...	3.2	0.87	...	2.6	0.59	...	2.9	0.42	...	2.3	0.19	...	2.2	0.16	...	1.0
571	0.06	...	....	0.06	...	....	0.15	...	....	0.23	...	....	0.41	...	....	0.42	...	....	0.33	...	....	0.33	...	....	0.19	...	....	0.12	...	....	0.07	...	....	0.09	...	....
572	0.95	...	4.1	0.79	...	2.0	0.83	...	2.6	0.57	...	2.8	0.07	...	0.6	0.04	...	0.3	0.02	...	0.1	Tr.	...	0.04	0.04	...	0.2	0.10	...	0.1	0.15	...	1.3	0.42	...	1.7
573	1.48	...	4.8	1.43	...	5.4	2.24	...	7.1	0.40	...	2.0	0.33	...	1.4	0.17	...	0.8	0.02	...	0.2	0.04	...	0.5	0.02	...	0.2	0.11	...	0.6	0.72	...	4.4	0.81	...	1.6
574	0.10	...	....	0.27	...	....	0.36	...	....	0.52	...	....	0.78	...	....	1.07	...	....	0.74	...	....	0.66	...	....	0.41	...	....	0.33	...	....	0.16	...	....	0.14	...	....
575	0.21	...	....	0.63	...	....	0.68	...	....	0.30	...	....	0.30	...	....	0.18	...	....	0.09	...	....	0.15	...	....	0.19	...	....	0.16	...	....	0.14	...	....	0.17	...	....
576	0.71	...	....	1.88	...	....	2.14	...	....	0.74	...	....	0.23	...	....	0.05	...	....	0.11	...	....	0.08	...	....	0.08	...	....	0.19	...	....	0.29	...	....	0.63	...	....
577	0.88	...	....	1.23	...	....	1.22	...	....	0.62	...	....	0.42	...	....	0.24	...	....	0.20	...	....	0.14	...	....	0.16	...	....	0.32	...	....	0.38	...	....	0.42	...	....
578	1.16	...	....	1.58	...	....	2.23	...	....	1.34	...	....	0.67	...	....	0.25	...	....	0.20	...	....	0.19	...	....	0.22	...	....	0.44	...	....	0.63	...	....	0.72	...	....
579	1.80	...	....	1.93	...	....	2.73	...	....	1.15	...	....	0.82	...	....	0.34	...	....	0.25	...	....	0.24	...	....	0.41	...	....	0.73	...	....	1.09	...	....	1.07	...	....
580	2.58	...	8.4	2.95	...	9.0	2.95	...	8.1	1.30	...	3.9	0.79	...	2.5	0.29	...	1.3	0.37	...	2.3	0.30	...	2.1	0.69	...	5.9	1.03	...	4.4	1.80	...	5.4	2.08	...	8.6
581	4.07	...	....	3.65	...	....	3.50	...	....	1.41	...	....	0.64	...	....	0.22	...	....	0.22	...	....	0.28	...	....	0.35	...	....	0.91	...	....	1.51	...	....	2.48	...	....
582	4.01	...	....	3.56	...	....	3.64	...	....	1.54	...	....	0.54	...	....	0.10	...	....	0.28	...	....	0.26	...	....	0.79	...	....	1.37	...	....	2.59	...	....	3.53	...	....
583	3.41	...	....	2.79	...	....	3.41	...	....	1.18	...	....	0.45	...	....	0.05	...	....	0.00	...	....	0.00	...	....	0.22	...	....	0.95	...	....	1.75	...	....	2.27	...	....
584	2.84	...	....	5.00	...	....	1.60	...	....	0.71	...	....	0.22	...	....	0.00	...	....	Tr.	...	....	0.01	...	....	0.01	...	....	0.39	...	....	0.76	...	....	1.05	...	....
585	3.06	...	12.2	3.00	...	7.1	3.13	...	9.5	1.64	...	5.5	0.20	...	1.7	0.03	...	0.3	0.06	...	1.2	0.08	...	1.8	0.06	...	0.6	0.36	...	1.5	0.81	...	4.4	1.96	...	6.1
586	0.10	...	0.8	0.06	...	0.5	0.10	...	0.6	0.03	...	0.3	0.04	...	0.4	0.02	...	0.2	0.01	...	0.1	0.04	...	0.2	0.02	...	0.1	0.04	...	0.2	0.02	...	0.3	0.14	...	0.6
587	5.57	...	20.3	4.58	...	11.2	3.46	...	9.4	1.94	...	7.7	0.30	...	2.1	0.06	...	0.6	0.00	...	0.0	0.02	...	0.2	0.08	...	0.8	0.23	...	1.1	1.27	...	3.9	3.67	...	8.3
588	0.09	...	....	0.18	...	....	0.36	...	....	0.62	...	....	Tr.	...	....	0.09	...	....	0.00	...	....	Tr.	...	....	Tr.	...	....	0.09	...	....	0.06	...	....	0.07	...	....
589	1.06	...	....	1.18	...	....	2.56	...	....	4.61	...	....	0.81	...	....	0.01	...	....	Tr.	...	....	0.02	...	....	0.08	...	....	0.28	...	....	1.22	...	....	0.87	...	....
590	4.75	...	....	3.11	...	....	6.74	...	....	8.79	...	....	2.09	...	....	0.08	...	....	0.04	...	....	0.18	...	....	0.39	...	....	2.49	...	....	7.08	...	....	6.49	...	....
591	4.62	...	....	3.34	...	....	6.59	...	....	4.98	...	....	1.46	...	....	0.00	...	....	0.37	...	....	1.65	...	....	5.37	...	....	8.31	...	....	8.84	...	....	7.06	...	....

TABLE 1—

Station No.	Station.	Elevation above sea, in feet.	Years of Record.			Maximum Observed Precipitation.					Annual Precipitation.			Observed Precipitation, in Percentages of Mean Annual Rainfall.										Note reference.
			For day or less.	For month and year.	Last year included.	Number of Minutes.			Day of 24 hours.	Month.	Minimum.	Mean.	Maximum.	Mean daily intensity on days of rainfall.	Maximum During Short Periods. Number of Minutes.			Mean for month of maximum record.	Maximum.		Annual.			
						5.	15.	60.							5.	15.	60.		In 24 hours.	In one calendar month.	Minimum.	Mean.	Maximum.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
592	Elisabethville, S. E. Congo			14	1925							45.2										100		15
593	Gazi, N. E. Congo			12	1922							72.7										100		15
594	Eala, N. Congo			16	1922							70.8										100		15
595	Boma, W. Congo			18	1922							35.7										100		15
596	St. Helena Island	1 983		29	1920					11.0	31.0	40.1	49.8					12.8		27.4	77	100	124	4
597	Calabar, Nigeria	42		24	1920					41.2	90.0	123.2	167.4					14.8		33.8	73	100	136	4
598	Lagos, Nigeria	7		29	1920					31.0	40.5	71.6	115.5					26.1		43.4	57	100	162	4
599	Akkra, Gold Coast	61		27	1920					21.1	13.1	27.2	44.2					25.8		77.9	48	100	163	4
600	Gambaga, W. Africa	350		20	1920					18.2	30.7	45.6	71.7					24.1		39.9	67	100	157	4
601	Timbuktu, W. Africa			15	1914							7.7										100		
602	Freetown, Sierra Leone	189		46	1920					61.1	102.5	157.2	206.6					23.8		38.9	65	100	132	4
603	Bathurst, Gambia	7		37	1920					36.6	24.6	47.6	80.8					41.1		77.0	52	100	170	4
604	La Laguna, Canary Islands	10		26	1920					20.6	10.3	23.4	41.7					16.3		87.8	44	100	178	4
605	Funchal, Madeira Islands	82		41	1920					19.0	10.7	27.2	55.8					14.6		69.6	39	100	204	4
606	Cape Spartel, Tangiers	193		28	1920					14.1	21.9	30.4	46.8					18.2		46.4	72	100	154	4
607	Algiers, Algeria	{1 129 { 13		27 54	1920 1894					11.0	12.7	27.4 26.9	42.7					14.6		40.2	46	100 100	158	4 15; 15
608	Tunis, Tunis	69		31	1920					7.1	11.0	16.5	22.8					11.3		43.0	67	100	138	4
609	Alexandria, Egypt	112	37	30	1922				11.0	15.4	3.2	8.0	23.0					22.6	137.5	192.7	40	100	288	4; 15
610	Cairo (Abbassia), Egypt	112	34	36	1922				1.4	2.0	0.2	1.3	3.4					21.5	107.8	154.0	18	100	261	4; 15
611	Aswan, Egypt	331		20	1920							0.0										100		19
612	Suez, Egypt	22	34	35	1920				1.2	2.5		1.0	3.6					12.0	120.0	250.0		100	360	19; 15
613	Ismaili, Suez Canal	39	16	35	1920				2.1	2.7		1.8	4.1	2.5				22.8	116.4	149.5		100	227	19
614	Gaza, Palestine	66		10	1905							16.5										100		15
615	Jerusalem, Palestine	1 968		{53 {44	1913 1895					16.4	13.4	25.2 25.5	45.0	1.7				21.8		65.0	53	100 100	178	15
616	Tiberias, Egypt	—653		18								18.1										100		15
617	Le Krey, Egypt	3 330		11	1914							59.6										100		15
618	Beirut, Syria	111		45	1920					15.7	23.2	35.5	51.1					16.2		43.2	65	100	144	4
619	Nicosia, Cyprus	525	14	14	1920				8.7			15.1						12.1	57.5			100		19
620	Kyrenia, Cyprus	46	14	14	1920				5.2			21.1						35.2	24.6			100		19
621	Candia, Crete	128	13	13	1920				4.4			21.1						20.1	20.8			100		19
622	Athens, Greece	356		26	1920					8.1	4.6	15.5	22.4+					16.8		52.0	30	100	144	4
623	Smyrna, Turkey			29	1882							25.6		1.4								100		20
624	Constantinople, Turkey	246		48	1895				6.3			28.9						5.7	21.7			100		15
625	Batak, S. W. Bulgaria	3 180		20	1918						13.9	26.6	36.5								53	100	127	14
626	Sofia, W. Bulgaria	1 803		20	1918						15.0	24.3	33.3								64	100	137	14
627	Petrohan, N. W. Bulgaria	4 584		15	1913						31.4	45.3	61.2								70	100	136	14
628	Ferdinand, N. W. Bulgaria	525		20	1918						8.1	21.4	31.8								38	100	148	14
669	Varna, N. E. Bulgaria	115	15	19	1918				3.9		14.8	20.3	26.2					8.3	19.2		78	100	129	14; 20
630	Sulina, E. Roumania	7		58	1925					10.3	5.2	15.4	24.4					8.3		67.0	84	100	188	4
631	Braila, E. Roumania	82	20	32	1910				3.8			17.9						7.5	21.2			100		22
632	Bucharest, E. Roumania	269		61	1924					11.7	13.5	23.1	34.0					15.0		50.8	58	100	147	4
633	Belgrade, Jugo-Slavia	452		35	1924					8.0	12.8	24.4	33.8					9.7		32.6	52	100	138	4
634	Lesina, S. W. Jugo-Slavia	66		61	1918					18.9	17.1	31.1	52.0					14.0		60.8	56	100	167	4
635	Obir, Austria	6 710		45	1923					21.1	36.1	60.2	91.5					5.7		35.0	60	100	152	4
636	Vienna, Austria	663		70	1920					9.0	16.5	25.5	35.6					10.7		35.2	65	100	140	4
637	Sonnblick, Austria	10 020		33	1923					15.5	41.2	65.0	79.7					7.9		23.9	63	100	123	4
637-A	Klagenfurt, Austria	1 476		88	1900					12.7	16.1	38.5	57.8					7.8		32.9	42	100	150	15; 23
638	Santis, Switzerland	8 200		38	1920					27.5	53.3	100.3	154.2					8.6		27.4	53	100	154	4
639	St. Gallen, Switzerland	2 230		50	1920							52.7										100		23
640	Zurich, Switzerland	1 565		57	1920					16.9	29.0	43.6	78.3					12.0		38.6	66	100	180	4
641	Chalet Capt, Switzerland	4 433		50	1900							81.7										100		9
642	Interlaken, Switzerland	1 862		26	1919							46.6										100		23
643	Geneva, Switzerland	1 329	30	50	1920				4.9			35.3						11.6	13.9			100		23; 9
644	Gotthard, Switzerland	6 890		13	1920							80.1										100		23
645	Locarno, Switzerland	785		20	1905							76.8										100		23
646	Domodossola, Italy	984		26	1905					25.3	32.8	58.3	73.6					13.1		45.6	80	100	138	23
647	Aosta, N. W. Italy	1 912		66	1910							22.4										100		23
648	Valdobria, N. W. Italy	8 356		20	1905					18.1	36.8	46.6	76.1					7.8		38.7	79	100	164	11
649	Milan, N. Italy	482		161	1924					14.8	24.9	39.4	61.8					12.0		37.6	68	100	157	4
650	Pavia, N. Italy	269		92	1908					11.3	23.0	31.1	51.0					10.3		36.3	74	100	164	23
651	Venice, N. E. Italy	154		26	1905					8.8	19.1	26.2	38.3					6.1		33.6	73	100	146	11
651-A	Padua, N. E. Italy	103	17	202	1926				3.5	12.3	18.9	33.8	61.6	1.0				9.8	10.7	36.4	56	100	182	15; 23
652	Camaldoli, N. Central Italy	3 680		21	1905					20.3	44.7	71.7	100.0					11.7		28.4	62	100	140	11
653	Florence, N. Central Italy	363		26	1905					10.7	15.9	33.0	44.8					11.8		22.4	48	100	136	11
654	Rome, Italy	208		143	1924					13.9	12.6	32.7	57.9					15.4		42.6	39	100	177	4
655	Naples, Italy	532		26	1905					10.3	17.4	33.6	44.4					12.1		30.6	52	100	132	11
656	Oppido Mamertina, S. Italy	1 228		24	1905					25.9	33.8	52.6	73.0					13.8		49.1	64	100	139	11
657	Catania, Sicily	213		32	1923					19.8	12.6	24.6	43.7					18.4		80.3	51	100	177	4
658	Sassari, Sardinia	735		42	1924					10.6	17.0	23.6	38.2					15.8		44.9	72	100	162	4
659	Genoa, Italy	259		26	1905					18.7	34.3	52.9	75.1					14.1		35.4	65	100	142	11
660	Nice, France	1 140	30	30	1910				5.7			32.3						18.4	17.6			100		9
661	Marseilles, France	246	30	50	1920				8.7	12.4	12.3	22.6	43.0					17.0	38.5	54.9	54	100	190	4; 9
662	Lyon, France	981	30	80	1920				4.3	11.3	19.3	29.1	45.2					12.7	14.8	38.8	66	100	156	4; 9
663	Besancon, France	1 020	30	30	1910				2.8			42.6						9.2	6.6			100		9
664	Nancy, France	724	30	30	1910				7.6			28.8						10.3	9.0			100		9
665	Paris, France	164	30	50	1923				2.0	6.2	10.9	22.7	29.8					10.1	8.8	27.3	48	100	131	4; 9
665-A	Paris, France	164	30	214	1926				2.0	6.9	9.0	19.9	29.8					9.5	10.0	34.6	45	100	150	15; 23
666	Clermont, France	1 309	30	30	1910				2.8			25.9						10.1	10.8			100		9
667	Puy de Dome, France	4 812	30	30	1910				6.5			66.7						9.9	9.8			100		9
668	Perpignan, S. E. France	106	30	50	1900				5.8	15.9	11.5	21.6	39.2					11.1	26.8	73.5	53	100	181	24; 9
669	La Preste, S. E. France	3 680		17	1895					18.7	25.0	44.3	62.1					5.1		35.5	56	100	140	24
670	Puymorens, S. E. France	6 320		28	1900					14.0	20.1	31.4	47.4					10.5		44.8	64	100	151	24
671	Huesca, N. E. Spain	1 653	20	39	1900				3.4	11.4	11.2	22.2	32.7					11.5	15.3	51.3	50	100	148	26
672	Saragossa, N. E. Spain	672	19	40	1899				2.6	6.5	7.0	12.0	22.3					11.5	21.7	54.1	58	100	186	26
673	Barcelona, Spain	70		30	1890					13.3	10.7	21.1	38.5					15.6		63.0	51	100	182	20
674	Palma, Spain			56	1921					9.3	9.4	18.9	30.6					11.6		49.2	50	100	162	4
675	Valencia, Spain	59		30	1890					13.8	7.5	19.1	50.7					15.6		72.0	39	100	265	20
676	Madrid, Spain	2 149		60	1919					7.3	10.2	16.6	27.5					9.9		44.0	61	100	166	4
677	Gibraltar, Gibraltar	53		67	1920					24.9	14.6	35.2	65.3					18.0		70.5	41	100	186	4
678	Seville, S. W. Spain	98		29	1890					12.3	6.3	18.6	34.7					11.7		66.0	34	100	186	20
679	Lisbon, Portugal	312		57	1920					15.1	17.3	29.3	55.9					13.9		51.4	59	100	190	4
680	Serra de Estrella, Portugal	4 756	19	19	1900				11.5	48.6	51.1	113.2	184.3	0.8				13.2	10.1	43.0	45	100	163	25
681	Oporto, Portugal	328	28	38	1900				6.5	20.4	23.8	51.4	89.0	0.8				10.6	12.7	39.8	46	100	178	20
682	Valladolid, Spain	2 345	20	40	1900				2.2	8.2	5.6	12.5	17.4	1.0				11.0	17.6	65.6	45	100	139	25
683	Oviedo, N. W. Spain	800	50	50	1900				3.2	16.4	25.2	36.0	49.8	1.4				11.5	8.9	45.5	70	100	138	25
684	Bilbao, N. Spain	56	20	46	1900				4.8	19.6	30.2	47.2	64.4					11.1	10.2	41.5	64	100	137	25
685	Pic du Midi, N. Spain	9 380		30	1910				4.9			65.5						7.5	7.5			100		9
686	Bayonne, S. W. France			30	1890							45.3										100		20
687	Toulouse, S. France	636		30	1910				3.6			26.1						12.3	13.8			100		9
688	Bordeaux, S. W. France	243	30	30	1910				2.5			30.3						9.2	8.2			100		9
689	Nantes, W. France	121	30	40	1920				2.2	10.0	22.1	30.9	41.1					12.1	7.1	32.3	73	100	133	4; 9
690	Brest, W. France	200	30	30	1910				2.8			32.3						9.4	8.7			100		9
691	Jersey Island	25	39	51	1921		1.59		2.4	10.8	15.9	33.5	45.6					12.2	7.2	32.2	47	100	136	25; 9
692	Arras, N. France	207	30	30	1910				2.1			26.5						9.6	7.9			100		9
693	Calais, N. France	73		38	1910							29.1										100		9
694	Dungeness, S. E. England	21	30	50	1921		1.60	2.8	2.4	10.4	13.6	24.4	46.6					11.3	9.8	42.6	56	100	191	25
695	Greenwich, England	150	81	81	1921			3.6	3.7	7.6	12.5	24.3	35.5					11.0	15.2	31.3	51	100	146	4; 25
696	London, Kew Obs., England	18	51	51	1921	0.7	1.4	3.28	2.4	7.2	12.2	23.8	38.2		2.9	5.9	13.7	9.0	10.1	30.1	51	100	160	15
697	Southampton, England	64	43	51	1921				2.9	11.1	16.1	30.9	43.3					12.8	9.4	36.0	52	100	140	25
698	Falmouth, S. W. England	167	51	51	1921				2.8	11.0	28.9	43.6	60.6					14.3	6.4	25.2	66	100	139	25
699	Scilly Island, England	131	41	61	1921				3.1	8.8	21.6	31.9	45.3					13.7	9.7	27.6	68	100	142	25
700	Cardiff, S. Wales	202		35	1915		1.02					41.4										100		25
701	Lluchfa, N. Wales	2 500		36	1917							161.0										100		13
702	Holyhead, England	15	46	51	1921				3.4	10.4	24.0	34.9	56.2					11.4	9.7	29.8	68	100	161	25

(Continued.)

Station No.	January			February			March			April			May			June			July			August			September			October			November			December		
	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
592	9.01	...	....	9.54	...	....	8.74	...	....	1.81	...	....	0.23	...	....	0.00	...	....	Tr.	...	....	0.01	...	....	0.14	...	....	1.18	...	....	4.42	...	....	10.11	...	....
593	2.65	...	....	3.38	...	....	5.24	...	....	8.25	...	....	7.24	...	....	5.48	...	....	5.73	...	....	8.30	...	....	8.52	...	....	7.95	...	....	8.46	...	....	4.00	...	....
594	3.75	...	....	4.94	...	....	4.92	...	....	5.50	...	....	7.23	...	....	4.80	...	....	4.02	...	....	5.56	...	....	7.36	...	....	8.76	...	....	6.83	...	....	6.74	...	....
595	4.39	...	....	3.11	...	....	5.57	...	....	7.27	...	....	2.18	...	....	0.01	...	....	0.01	...	....	0.15	...	....	0.43	...	....	2.02	...	....	5.40	...	....	5.07	...	....
596	1.97	...	5.7	3.80	...	8.9	4.94	...	11.0	3.87	...	10.1	4.15	...	6.7	4.07	...	7.9	4.02	...	8.3	3.72	...	7.7	3.02	...	5.8	1.91	...	4.3	1.68	...	3.9	1.96	...	4.2
597	1.70	...	10.6	2.22	...	8.5	6.22	...	16.4	8.57	...	14.7	12.41	...	23.7	16.24	...	22.6	19.47	...	34.3	18.27	...	41.2	16.61	...	23.8	11.96	...	20.3	7.59	...	14.3	1.96	...	7.2
598	1.07	...	8.1	2.07	...	12.5	3.74	...	11.3	5.75	...	12.5	10.47	...	21.1	18.65	...	31.0	10.68	...	29.9	2.80	...	22.5	5.36	...	15.9	7.76	...	16.5	2.58	...	7.0	0.80	...	4.4
599	0.65	...	2.5	1.00	...	6.3	1.83	...	8.8	3.70	...	8.8	5.65	...	13.6	7.00	...	21.1	1.70	...	8.9	0.61	...	4.1	0.98	...	5.2	1.94	...	9.2	1.49	...	8.9	0.69	...	3.2
600	0.08	...	0.9	0.05	...	0.7	0.68	...	2.0	2.35	...	5.3	5.02	...	10.5	5.68	...	16.6	7.44	...	12.7	10.97	...	18.2	9.46	...	14.6	2.92	...	6.2	0.54	...	1.9	0.20	...	1.5
601	0.00	...	....	0.01	...	....	0.05	...	....	Tr.	...	....	0.24	...	....	0.80	...	....	2.79	...	....	2.81	...	....	0.36	...	....	0.15	...	....	Tr.	...	....	0.00	...	....
602	0.41	...	4.1	0.30	...	4.2	1.16	...	5.8	4.06	...	13.2	11.47	...	28.5	20.04	...	48.0	35.58	...	57.2	36.57	...	61.1	28.46	...	48.0	12.62	...	32.9	5.12	...	11.8	1.42	...	7.2
603	0.008	...	0.1	0.013	...	0.2	0.002	...	0.1	0.014	...	0.4	0.16	...	2.2	2.95	...	10.0	10.91	...	27.4	19.60	...	36.6	10.05	...	19.8	3.66	...	10.6	0.20	...	2.1	0.08	...	2.7
604	4.10	...	9.9	3.45	...	18.3	2.91	...	9.1	1.76	...	6.9	0.56	...	2.2	0.20	...	1.5	0.16	...	1.1	0.09	...	0.7	0.50	...	1.7	2.38	...	5.5	3.74	...	7.4	3.83	...	20.6
605	3.24	...	9.3	3.59	...	17.5	3.41	...	12.8	1.94	...	9.7	1.09	...	4.9	0.37	...	2.5	0.13	...	1.4	0.13	...	1.2	1.19	...	6.2	3.97	...	19.0	4.68	...	18.9	3.22	...	17.4
606	3.97	...	11.8	3.36	...	8.9	4.10	...	9.1	2.17	...	6.4	1.70	...	5.2	0.52	...	1.8	0.07	...	1.0	0.07	...	0.9	1.06	...	5.2	3.04	...	5.7	5.52	...	14.1	4.78	...	11.2
607	4.02	...	11.0	2.62	...	4.6	3.31	...	8.9	2.09	...	5.0	1.73	...	6.7	0.74	...	2.5	0.13	...	1.6	0.12	...	0.9	1.17	...	4.1	3.41	...	8.2	4.11	...	10.6	4.00	...	7.7
	3.82	...	....	3.31	...	....	3.12	...	....	2.01	...	....	1.30	...	....	0.59	...	....	0.08	...	....	0.28	...	....	1.02	...	....	2.88	...	....	3.91	...	....	4.60	...	....
608	2.07	...	6.3	2.01	...	5.3	1.87	...	7.1	1.48	...	4.7	0.91	...	2.8	0.50	...	4.5	0.11	...	0.7	0.21	...	1.2	0.98	...	4.8	1.86	...	5.5	2.13	...	5.7	2.38	...	6.4
609	2.13	1.8	5.0	0.91	1.3	1.9	0.55	0.6	2.1	0.12	0.4	2.0	0.04	0.4	0.4	Tr.	0.1	0.1	0.00	Tr.	0.0	Tr.	0.1	0.1	0.04	0.6	0.8	0.24	1.1	2.3	1.38	1.8	4.0	2.60	11.0	13.4
610	0.36	1.4	1.7	0.20	1.0	1.1	0.16	0.6	0.8	0.12	1.2	1.5	0.04	0.3	0.5	0.01	0.05	0.3	0.00	0.0	0.0	0.00	0.0	0.0	0.00	Tr.	0.0	0.08	0.9	1.2	0.12	0.6	1.1	0.28	0.6	3.0
611	0.00	...	....	0.00	...	....	0.00	...	....	0.00	...	....	0.00	...	....	0.00	...	....	0.00	...	....	0.00	...	....	0.00	...	....	0.00	...	....	0.00	...	....	0.00	...	....
612	0.24	0.8	2.1	0.12	0.5	0.8	0.12	0.7	0.9	0.12	0.6	1.2	0.12	1.3	2.5	Tr.	0.2	0.2	0.00	0.0	0.0	0.00	0.0	0.0	Tr.	0.1	0.1	0.04	0.4	0.5	0.12	0.4	0.4	0.16	1.1	1.8
613	0.43	2.1	2.7	0.24	0.3	1.0	0.24	0.4	0.9	0.20	1.5	1.8	0.12	0.4	1.6	0.00	0.0	0.1	0.00	0.0	0.0	Tr.	Tr.	Tr.	Tr.	0.1	0.2	0.08	0.3	1.7	0.16	1.1	1.6	0.22	0.3	1.4
614	4.34	...	....	1.97	...	....	1.46	...	....	0.28	...	....	0.31	...	....	0.04	...	....	0.00	...	....	0.00	...	....	0.04	...	....	0.82	...	....	3.15	...	....	4.13	...	....
615	6.38	...	14.4	5.06	...	12.2	3.56	...	13.3	1.69	...	6.5	0.27	...	1.2	0.01	...	....	0.00	...	....	0.00	...	....	0.04	...	0.8	0.41	...	3.0	2.29	...	8.0	5.50	...	10.4
	6.30	...	....	5.76	...	....	3.57	...	....	1.73	...	....	0.28	...	....	0.00	...	....	0.00	...	....	0.00	...	....	0.04	...	....	0.39	...	....	2.05	...	....	5.35	...	....
616	4.52	...	....	2.90	...	....	2.36	...	....	0.90	...	....	0.15	...	....	0.00	...	....	0.00	...	....	0.00	...	....	0.00	...	....	0.55	...	....	2.38	...	....	4.33	...	....
617	12.81	...	....	10.86	...	....	9.19	...	....	4.02	...	....	1.91	...	....	0.11	...	....	0.01	...	....	0.00	...	....	0.30	...	....	2.45	...	....	7.42	...	....	10.54	...	....
618	7.32	...	14.9	5.75	...	15.7	3.92	...	9.5	2.23	...	9.4	0.84	...	9.3	0.14	...	2.7	0.02	...	0.5	0.03	...	0.3	0.30	...	2.4	2.10	...	8.1	5.28	...	15.2	7.84	...	18.4
619	2.59	1.8	....	1.98	1.3	....	1.73	1.9	....	0.82	2.6	....	1.00	1.2	....	0.32	1.8	....	0.01	0.1	....	0.00	0.0	....	0.35	1.8	....	0.88	1.0	....	1.63	3.7	....	3.40	4.5	....
620	4.36	2.8	....	3.50	1.9	....	2.29	2.3	....	0.77	1.3	....	0.77	0.9	....	0.22	1.7	....	0.00	0.0	....	0.00	0.0	....	0.28	1.2	....	1.25	1.7	....	2.38	3.9	....	5.33	5.2	....
621	3.57	1.7	....	3.54	2.2	....	1.75	1.3	....	1.01	0.9	....	0.80	1.6	....	0.09	0.3	....	0.04	0.5	....	0.14	1.3	....	0.64	3.9	....	1.81	2.2	....	3.42	2.6	...	4.25	4.4	....
622	2.06	...	5.5	1.66	...	4.0	1.16	...	2.3	0.66	...	2.7	0.83	...	3.0	0.69	...	4.7	0.28	...	2.0	0.49	...	2.9	0.66	...	2.3	1.87	...	4.6	2.61	...	8.1	2.63	...	5.8
623	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	0.12	...	....	....	...	....	....	...	....	....	...	....	4.40	...	....
624	3.42	1.3	....	2.72	1.4	....	2.44	1.8	....	1.65	1.2	....	1.18	1.4	....	1.34	1.6	....	1.06	1.9	....	1.65	6.3	....	2.05	3.3	....	2.52	3.4	....	4.02	2.2	....	4.90	1.6	....
625	1.89	...	....	1.70	...	....	2.01	...	....	2.33	...	....	2.92	...	....	3.58	...	....	3.00	...	....	1.97	...	....	1.78	...	....	1.74	...	....	2.08	...	....	1.70	...	....
626	1.26	...	....	1.18	...	....	1.64	...	....	1.97	...	....	2.96	...	....	3.08	...	....	2.56	...	....	2.05	...	....	2.41	...	....	2.09	...	....	2.12	...	....	1.10	...	....
627	2.37	...	....	3.00	...	....	3.64	...	....	3.97	...	....	4.84	...	....	5.56	...	....	4.64	...	....	3.27	...	....	4.14	...	....	3.74	...	....	3.78	...	....	2.48	...	....
628	1.22	...	....	1.02	...	....	1.38	...	....	1.46	...	....	2.83	...	....	2.76	...	....	2.29	...	....	1.86	...	....	1.86	...	....	2.21	...	....	1.89	...	....	1.06	...	....
629	1.37	1.1	....	1.14	1.7	....	1.14	0.9	....	1.34	1.3	....	1.58	1.5	....	2.72	1.6	....	1.70	3.9	....	1.66	2.9	....	1.42	3.5	....	2.01	2.2	....	2.01	2.6	....	2.22	1.3	....
630	0.99	...	3.2	0.84	...	2.1	1.08	...	4.1	1.15	...	4.6	1.24	...	4.7	1.90	...	6.7	1.42	...	5.1	1.28	...	10.3	1.38	...	5.0	1.52	...	4.7	1.49	...	5.0	1.08	...	3.7
631	1.26	2.2	....	0.98	0.9	....	1.25	3.8	....	1.37	1.4	....	1.79	1.1	....	2.80	3.4	....	1.99	2.9	....	1.12	1.9	....	1.28	1.5	....	1.46	1.1	....	1.19	2.2	....	1.30	1.0	....
632	1.34	...	4.5	1.09	...	5.8	1.65	...	4.7	1.73	...	5.2	2.47	...	6.8	3.46	...	11.7	2.68	...	7.7	2.00	...	7.0	1.56	...	6.7	1.70	...	5.5	1.90	...	6.3	1.59	...	5.3
633	1.31	...	3.9	1.19	...	3.5	1.62	...	4.2	2.39	...	5.2	2.75	...	7.6	3.00	...	5.8	2.59	...	7.5	1.92	...	6.0	1.71	...	4.1	2.36	...	8.0	1.92	...	6.4	1.61	...	3.2
634	3.05	...	7.9	2.51	...	9.5	2.87	...	6.8	2.37	...	8.2	1.59	...	4.8	1.51	...	3.9	0.80	...	3.9	1.51	...	12.7	2.50	...	12.0	3.87	...	13.8	4.36	...	18.9	4.14	...	11.1
635	3.07	...	7.0	2.43	...	21.1	4.61	...	14.1	5.24	...	12.1	5.43	...	16.4	6.61	...	12.7	6.65	...	12.7	6.38	...	13.7	5.47	...	18.9	5.91	...	13.0	4.06	...	11.4	3.39	...	9.0
636	1.46	...	5.0	1.80	...	5.2	1.81	...	4.8	2.05	...	6.3	2.80	...	7.3	2.72	...	9.0	3.11	...	8.1	2.72	...	7.2	1.97	...	4.8	1.88	...	5.2	1.77	...	5.4	1.81	...	5.4
637	4.90	...	8.7	4.90	...	9.6	6.23	...	13.7	6.88	...	11.9	6.18	...	13.4	5.51	...	10.1	5.62	...	13.5	5.08	...	9.2	4.66	...	8.4	5.18	...	15.5	4.05	...	11.0	5.35	...	12.8
637-A	1.50	...	6.5	1.34	...	5.9	1.97	...	8.9	2.56	...	8.6	3.74	...	10.2	4.33	...	10.9	4.72	...	11.2	4.92	...	9.3	3.62	...	9.7	4.08	...	12.0	2.99	...	12.7	1.98	...	7.7
638	7.08	...	22.6	6.25	...	21.0	7.07	...	27.0	8.38	...	18.7	7.73	...	17.9	10.85	...	20.8	12.03	...	23.7	10.87	...	24.4	8.61	...	18.1	6.68	...	16.8	5.85	...	18.7	8.67	...	27.5
639	2.48	...	....	2.48	...	....	3.34	...	....	4.48	...	....	5.39	...	....	7.05	...	....	6.76	...	....	6.10	...	....	5.67	...	....	4.05	...	....	2.92	...	....	2.99	...	....
640	2.06	...	6.9	2.15	...	7.5	2.96	...	11.3	3.53	...	8.1	4.20	...	10.2	5.24	...	16.9	5.11	...	10.9	5.07	...	9.6	4.04	...	9.7	3.58	...	8.5	2.67	...	7.7	3.01	...	7.1
641	5.91	...	....	5.32	...	....	6.85	...	....	6.30	...	....	7.28	...	....	8.20	...	....	6.76	...	....	7.12	...	....	6.62	...	....	8.35	...	....	6.38	...	....	6.62	...	....
642	2.80	...	....	2.56	...	....	3.71	...	....	3.63	...	....	4.10	...	....	5.40	...	....	5.68	...	....	4.85	...	....	3.99	...	....	3.38	...	....	2.72	...	....	3.86	...	....
643	1.69	1.8	....	1.97	2.0	....	2.52	1.8	....	2.75	1.8	....	2.87	1.2	....	3.27	2.0	....	3.19	1.8	....	3.77	2.3	....	3.19	1.9	....	4.10	4.9	....	3.19	2.4	....	2.72	1.2	....
644	5.08	...	....	3.86	...	....	7.25	...	....	6.73	...	....	6.69	...	....	7.42	...	....	7.17	...	....	7.00	...	....	9.25	...	....	8.20	...	....	5.38	...	....	6.14	...	....
645	3.84	...	....	2.78	...	....	5.17	...	....	6.85	...	....	8.88	...	....	7.17	...	....	7.37	...	....	9.21	...	....	8.59	...	....	9.60	...	....	5.71	...	....	3.27	...	....
646	3.16	...	9.1	2.82	...	10.6	3.85	...	13.4	5.40	...	14.0	6.60	...	14.9	4.76	...	12.2	3.61	...	7.8	4.65	...	12.4	5.25	...	18.6	7.21	...	25.3	4.78	...	17.1	3.39	...	8.5
647	1.82	...	....	1.89	...	....	1.77	...	....	1.54	...	....	2.48	...	....	1.50	...	....	1.14	...	....	1.58	...	....	1.61	...	....	3.06	...	....	2.12	...	....	1.77	...	....
648	4.00	...	14.2	3.52	...	16.6	7.10	...	16.9	5.60	...	10.4	5.02	...	10.0	3.14	...	8.0	2.30	...	5.1	2.12	...	5.5	2.87	...	9.6	3.64	...	18.1	3.72	...	11.2	3.95	...	11.0
649	2.43	...	8.2	2.26	...	10.3	2.80	...	10.4	3.41	...	11.1	3.86	...	13.4	3.24	...	10.6	2.81	...	9.2	2.15	...	8.9	3.44	...	13.1	4.74	...	14.8	4.22	...	13.7	3.05	...	12.7
650	2.13	...	6.5	1.97	...	6.2	2.25	...	8.1	2.96	...	7.8	3.20	...	11.3	2.25	...	8.3	2.01	...	6.4	1.97	...	7.0	2.60	...	10.7	2.99	...	10.4	3.85	...	8.9	2.46	...	7.1
651	1.46	...	4.6	1.60	...	8.8	2.18	...	5.2	2.23	...	4.3	2.90	...	8.4	2.90	...	5.8	1.92	...	4.6	2.04	...	5.4	2.55	...	7.1	2.75	...	8.2	2.18	...	7.4	1.60	...	3.8
651-A	2.12	1.5	12.1	1.88	1.6	7.3	2.44	1.7	7.7	2.97	2.2	9.1	3.32	1.7	12.3	3.42	2.2	9.2	3.56	2.0	8.3	2.55	2.0	10.7	2.96	2.4	9.5	3.78	3.5	12.0	3.25	2.2	11.0	2.45	1.4	12.0
652	6.82	...	17.3	5.16	...	18.0	6.43	...	17.7	7.95	...	15.8	7.08	...	11.7	5.01	...	10.2	3.86	...	7.1	2.64	...	8.1	3.84	...	7.2	9.22	...	16.4	8.40	...	20.2	6.68	...	17.5
653	2.01	...	6.6	2.01	...	5.9	2.80	...	6.6	2.91	...	6.4	3.14	...	7.8	2.66	...	6.1	1.48	...	6.0	1.98	...	6.4	3.27	...	10.1	4.06	...	9.1	3.88	...	10.7	2.82	...	6.8
654	3.25	...	8.9	2.67	...	8.6	2.87	...	7.8	2.60	...	9.0	2.18	...	6.8	1.56	...	5.5	0.89	...	4.5	1.04	...	4.2	2.54	...	10.2	5.04	...	13.9	4.41	...	13.6	3.87	...	10.3
655	3.66	...	7.6	2.69	...	6.0	2.65	...	7.4	3.06	...	6.5	2.30	...	4.3	1.38	...	5.7	0.50	...	2.2	0.87	...	2.9	2.90	...	6.2	4.98	...	8.2	4.06	...	10.2	4.60	...	9.6
656	7.06	...	25.9	5.22	...	12.9	5.19	...	14.7	4.73	...	10.5	3.38	...	6.3	1.65	...	5.2	0.78	...	4.8	1.28	...	3.8	2.80	...	6.2	6.72	...	20.5	6.16	...	13.8	7.88	...	17.0
657	2.62	...	12.5	2.38	...	7.9	1.95	...	7.4	1.54	...	5.0	0.76	...	3.2	0.24	...	1.4	0.15	...	1.1	0.56	...	7.4	1.67	...	15.8	3.24	...	11.9	4.54	...	19.8	3.06	...	9.8
658	2.37	...	6.7	1.97	...	5.7	2.24	...	5.3	2.16	...	5.3	1.70	...	5.4	0.93	...	3.5	0.35	...	1.5	0.40	...	3.0	1.51	...	4.5	3.29	...	8.8	3.78	...	10.6	2.91	...	7.3
659	4.24	...	10.9	4.14	...	12.8	4.65	...	15.8	4.35	...	10.0	3.64	...	7.4	2.52	...	9.8	2.06	...	8.3	1.98	...	7.1	4.92	...	17.3	7.06	...	14.9	7.42	...	18.7	4.81	...	14.6
660	2.08	2.8	....	2.14	2.9	....	2.94	4.3	....	2.61	2.5	....	2.40	2.9	....	2.07	3.5	....	1.00	2.8	....	1.01	2.4	....	2.22	3.6	....	5.96	5.7	....	4.53	8.8	....	3.34	3.8	....
661	1.60	1.8	4.8	1.44	2.3	5.1	1.65	2.0	5.6	2.16	1.9	6.6	1.73	2.9	6.9	1.12	1.8	3.8	0.65	4.1	4.0	0.83	2.9	4.8	2.41	4.0	11.1	3.85	8.7	12.4	2.85	5.9	10.8	2.11	3.6	9.4
662	1.34	0.9	4.9	1.36	2.4	4.9	1.68	2.2	4.9	2.40	2.2	6.3	2.82	3.1	10.0	3.01	2.8	8.1	2.68	2.8	6.1	2.85	4.1	9.6	2.81	3.4	8.3	3.70	4.2	11.3	2.43	3.4	7.3	1.80	1.5	5.6
663	2.71	1.8	....	2.76	1.4	....	3.00	2.2	....	3.06	1.9	....	3.64	2.4	....	3.91	2.8	....	3.98	2.1	....	3.50	2.1	....	3.98	1.9	....	4.62	2.1	....	3.54	3.2	....	3.80	1.9	....
664	1.91	1.2	....	1.86	0.9	....	1.97	1.0	....	1.84	1.1	....	2.27	1.3	....	2.80	2.4	....	2.79	1.6	....	2.38	1.9	....	2.53	2.4	....	2.96	2.6	....	2.66	1.4	....	2.87	1.2	....
665	1.52	0.9	2.9	1.35	1.0	3.5	1.63	1.1	3.6	1.67	1.1	4.0	1.89	1.9	3.7	2.11	1.5	4.2	2.19	1.3	4.7	2.07	1.6	4.9	1.91	1.2	4.7	2.30	2.0	6.2	1.89	1.1	4.3	2.04	0.7	5.3
665-A	1.36	0.9	3.5	1.19	1.0	4.5	1.31	1.1	3.6	1.52	1.1	4.5	1.83	1.9	4.6	1.98	1.5	6.7	2.10	1.3	5.4	1.90	1.6	6.9	1.74	1.2	4.7	1.84	2.0	6.2	1.63	1.1	4.8	1.56	0.7	5.4
666	1.25	1.4	....	1.51	1.3	....	1.72	1.9	....	2.08	2.1	....	2.65	1.8	....	2.90	1.9	....	2.62	1.8	....	2.61	2.8	....	2.92	2.2	....	2.22	1.7	....	1.75	1.5	....	1.68	1.6	....
667	6.25	3.5	....	6.01	3.9	....	6.59	6.5	....	5.50	3.2	....	4.82	2.2	....	5.08	3.0	....	5.08	2.7	....	4.90	3.9	....	5.23	4.3	....	5.61	2.4	....	5.22	2.4	....	6.40	3.0	....
668	2.36	4.9	10.0	1.74	2.2	7.2	1.97	3.5	7.0	1.85	2.1	6.4	2.26	2.1	8.8	1.55	3.2	5.3	0.79	2.0	4.0	1.06	2.1	5.3	1.75	4.5	7.2	2.39	2.3	15.9	2.22	3.6	10.4	1.63	5.8	7.6
669	3.40	...	13.6	2.25	...	15.7	3.66	...	8.8	4.44	...	8.3	5.56	...	13.5	4.55	...	7.5	5.41	...	14.1	4.10	...	8.8	3.74	...	6.9	3.48	...	13.1	3.32	...	7.3	2.42	...	9.4
670	3.31	...	14.0	2.18	...	4.9	2.46	...	5.2	2.72	...	5.9	2.62	...	5.1	3.06	...	7.0	2.50	...	6.7	2.28	...	7.0	2.24	...	6.4	2.74	...	5.2	2.46	...	5.3	2.88	...	9.0
671	1.28	1.3	4.8	1.46	2.3	4.6	2.01	1.3	5.5	2.16	2.6	7.3	2.60	2.3	6.6	1.70	2.2	8.5	1.26	1.7	6.4	1.38	3.4	7.1	2.56	3.4	11.4	2.52	3.1	7.3	2.05	1.8	5.6	1.19	2.8	3.9
672	0.79	0.6	2.5	0.75	1.4	3.6	1.02	1.2	3.4	1.18	1.2	6.0	1.66	1.3	4.4	1.04	1.8	4.9	0.71	1.0	3.4	0.55	1.3	2.4	0.94	1.8	4.4	1.36	2.6	6.5	1.22	1.6	4.8	0.71	1.0	2.2
673	1.30	...	3.7	1.46	...	4.0	1.74	...	4.7	2.06	...	7.0	1.30	...	4.2	1.34	...	5.1	0.87	...	3.0	1.38	...	5.6	3.38	...	13.1	2.30	...	13.3	1.34	...	4.7	1.69	...	12.0
674	1.61	...	7.2	1.39	...	3.8	1.55	...	4.6	1.41	...	4.5	1.57	...	5.2	0.71	...	3.7	0.42	...	1.9	0.60	...	4.6	2.04	...	6.4	3.12	...	7.2	2.20	...	9.2	2.22	...	8.0
675	1.84	...	3.7	1.22	...	7.9	1.54	...	7.2	1.54	...	4.7	1.70	...	10.0	0.83	...	7.2	0.47	...	2.2	0.35	...	2.8	2.99	...	18.8	3.21	...	8.7	1.97	...	13.8	1.88	...	7.8
676	1.38	...	5.6	1.27	...	5.6	1.59	...	6.1	1.65	...	7.3	1.67	...	4.2	1.34	...	4.2	0.38	...	4.7	0.48	...	3.1	1.46	...	5.5	1.80	...	4.0	2.00	...	5.1	1.32	...	5.1
677	5.08	...	21.6	4.22	...	14.2	4.79	...	17.8	2.68	...	8.0	1.73	...	6.5	0.48	...	3.9	0.03	...	0.6	0.10	...	1.7	1.39	...	5.5	3.31	...	12.4	6.38	...	24.9	5.52	...	23.8
678	2.17	...	12.3	1.81	...	7.1	2.50	...	9.1	1.89	...	9.3	1.65	...	6.5	0.59	...	4.0	0.04	...	0.3	0.16	...	1.7	0.75	...	4.6	1.88	...	6.4	2.40	...	10.4	2.72	...	8.1
679	3.63	...	11.5	2.49	...	9.8	3.44	...	8.0	2.59	...	8.9	1.95	...	5.2	0.70	...	3.1	0.17	...	2.2	0.24	...	2.0	1.44	...	8.8	3.26	...	10.3	4.30	...	9.7	4.08	...	15.1
680	12.50	10.8	27.0	9.20	6.9	32.5	15.00	11.5	48.6	9.96	7.4	30.7	9.50	3.2	17.4	4.85	2.8	8.5	1.38	2.9	6.0	1.38	2.3	3.2	6.15	7.4	24.5	14.38	8.0	30.1	15.90	9.3	39.3	11.41	6.9	34.1
681	6.41	3.7	20.1	4.95	3.6	16.2	5.44	3.1	16.0	4.66	2.2	12.2	4.10	2.4	13.4	2.05	3.0	10.4	0.94	1.5	4.2	1.06	3.7	7.8	2.99	3.9	15.0	6.50	4.6	19.2	6.92	4.5	17.0	5.62	3.4	20.4
682	0.90	0.9	3.9	0.91	1.7	4.0	1.10	1.2	4.4	1.06	0.9	3.0	1.62	2.2	3.5	1.10	1.1	3.8	0.35	0.6	2.0	0.35	0.9	1.4	1.30	1.4	3.8	1.34	1.2	3.3	1.38	1.0	5.3	0.98	1.5	2.8
683	2.19	3.2	10.0	2.48	1.6	7.1	4.14	1.8	16.4	3.58	1.9	8.0	2.76	1.5	8.2	2.48	2.0	6.8	2.01	2.2	5.4	1.62	1.3	5.9	2.68	2.0	8.3	3.54	1.5	8.5	3.70	1.7	8.7	3.74	1.5	12.7
684	4.40	1.5	10.9	3.39	1.8	10.8	4.65	1.9	14.2	3.94	2.3	10.1	3.19	1.9	7.8	3.27	3.4	9.0	2.37	3.4	10.2	2.17	4.8	8.9	3.75	1.8	15.7	5.25	2.3	13.7	5.68	2.2	13.1	5.25	2.5	19.8
685	7.46	4.2	....	6.34	3.4	....	6.11	2.6	....	6.98	2.5	....	5.06	2.6	....	4.92	4.9	....	3.31	4.5	....	3.29	3.4	....	3.96	3.7	....	4.70	3.9	....	5.83	3.9	....	7.61	3.9	....
686	3.66	...	....	2.98	...	....	3.76	...	....	2.74	...	....	2.95	...	....	3.23	...	....	2.35	...	....	2.72	...	....	4.50	...	....	5.55	...	....	3.74	...	....	4.16	...	....
687	1.78	1.2	....	1.63	1.5	....	1.84	1.5	....	2.70	2.6	....	2.78	2.8	....	3.21	3.0	....	1.81	2.2	....	1.90	2.9	....	2.00	2.8	....	2.22	1.6	....	2.21	1.6	....	2.07	2.0	....
688	2.12	1.2	....	2.05	1.2	....	2.18	1.3	....	2.60	1.8	....	2.74	2.1	....	2.80	2.5	....	1.96	1.4	....	1.84	2.2	....	2.33	1.6	....	3.41	1.4	....	3.13	1.3	....	3.14	1.3	....
689	2.82	1.1	8.0	2.30	1.4	6.4	2.49	1.2	6.2	1.99	1.1	4.3	2.28	1.0	4.9	2.24	2.1	6.1	2.19	1.4	4.7	1.85	1.5	5.0	2.12	1.3	4.9	3.76	2.2	8.6	3.44	1.3	8.9	3.72	1.8	10.0
690	3.06	2.8	....	3.09	1.0	....	2.12	1.2	....	2.35	1.7	....	1.64	1.3	....	1.85	2.0	....	1.94	1.4	....	2.15	1.5	....	2.46	1.8	....	4.01	2.7	....	3.98	1.8	....	3.70	1.8	....
691	2.94	1.3	6.4	2.44	1.2	6.6	2.51	1.3	6.6	1.94	1.1	4.8	1.82	0.8	4.6	2.11	1.9	4.9	2.10	1.7	7.1	2.44	2.2	7.7	2.41	1.9	8.0	4.55	2.4	10.1	4.08	1.5	10.8	4.15	1.8	7.9
692	1.92	0.9	....	1.73	0.9	....	1.97	1.2	....	1.62	0.9	....	2.14	1.4	....	2.50	1.8	....	2.55	2.1	....	2.33	1.5	....	2.14	1.2	....	2.64	1.2	....	2.33	1.1	....	2.56	1.1	....
693	2.28	...	....	1.73	...	....	1.73	...	....	1.57	...	....	2.06	...	....	2.05	...	....	3.20	...	....	2.72	...	....	2.84	...	....	4.06	...	....	3.11	...	....	2.70	...	....
694	1.90	1.9	5.5	1.60	1.3	4.4	1.76	1.9	5.1	1.36	1.0	4.6	1.30	1.3	3.8	1.66	2.0	5.9	1.81	1.2	4.8	1.98	1.8	5.4	2.05	1.5	5.2	3.52	2.4	8.4	2.75	1.1	10.4	2.17	1.1	7.6
695	1.86	1.6	4.4	1.52	0.8	4.0	1.62	1.5	4.1	1.61	2.5	4.3	1.86	1.6	4.4	2.01	1.9	6.1	2.42	2.7	7.3	2.34	1.8	5.4	2.07	1.5	5.5	2.68	2.6	7.6	2.23	1.3	6.0	2.00	1.2	6.0
696	1.76	1.6	4.9	1.54	0.9	4.1	1.69	0.9	3.9	1.45	1.2	4.0	1.72	1.8	4.1	2.15	2.4	7.2	2.17	2.8	4.9	2.24	1.8	6.8	1.87	1.6	5.7	2.70	1.4	6.9	2.72	1.3	6.0	2.23	1.5	6.4
697	2.67	1.7	8.2	2.30	1.1	5.7	2.28	1.3	5.3	1.85	1.2	4.3	2.00	1.2	4.5	2.01	2.5	5.4	2.28	2.1	6.2	2.62	2.4	6.9	2.18	2.0	6.7	3.94	1.9	11.1	3.14	1.7	6.3	3.66	2.9	9.9
698	4.21	1.7	9.7	3.70	2.1	8.7	3.46	1.6	8.3	2.64	1.5	7.0	2.21	1.1	5.8	2.31	1.7	7.5	2.84	1.8	6.2	3.25	2.8	10.5	2.91	1.9	8.1	4.96	2.6	10.7	4.80	2.4	8.5	6.27	3.3	11.8
699	2.96	2.8	6.7	2.57	1.5	7.1	2.45	1.6	5.9	1.91	0.9	4.2	1.67	1.2	4.2	1.71	1.3	5.6	2.22	1.6	5.2	2.62	2.0	8.1	2.45	1.8	7.8	3.66	1.6	7.2	3.27	3.1	6.7	4.29	1.3	8.8
700	3.09	...	....	2.94	...	....	3.15	...	....	2.50	...	....	2.45	...	....	2.51	...	....	3.08	...	....	4.23	...	....	3.05	...	....	4.75	...	....	4.08	...	....	5.01	...	....
701	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....	....	...	....
702	2.91	1.3	7.5	2.44	1.4	6.0	2.62	1.5	7.1	2.08	1.2	3.8	1.96	1.2	4.3	2.15	1.1	5.0	2.61	3.4	7.8	3.18	2.7	8.9	2.68	2.2	7.4	3.99	2.9	10.4	4.14	2.7	8.0	4.16	1.5	8.1

TABLE 1.—

Station No.	Station.	Elevation above sea, in feet.	Years of Record.			Maximum Observed Precipitation.					Annual Precipitation.			Observed Precipitation, in Percentages of Mean Annual Rainfall.										Data references.
			For day or less.	For month and year.	Last year included.	Number of Minutes.			Day of 24 hours.	Month.	Minimum.	Mean.	Maximum.	Mean daily intensity on days of rainfall.	Maximum During Short Periods.			Mean for month of maximum record.	Maximum.		Annual.			
						5.	15.	60.							Number of Minutes.				In 24 hours.	In one calendar month.	Minimum.	Mean.	Maximum.	
															5.	15.	60.							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
703	Dublin, Ireland	47	51	51	1921				3.4	7.6	16.6	27.4	38.5					11.1	12.4	27.7	61	100	140	25
704	Cork Harbor, Ireland	32		35	1915							41.9										100		25
705	Valencia, S. W. Ireland	30	52	52	1921				2.8	11.4	42.5	56.8	69.1					11.5	4.9	18.4	75	100	122	25; 4
706	Killarney, Ireland	174		35	1915							54.7										100		25
707	Kilkenny, Ireland	212		35	1915							33.1										100		25
708	Mullingar, Ireland	360		35	1915							36.3										100		25
709	Crossmolina, N. W. Ireland	67		35	1915							53.3										100		25
710	Armagh, N. E. Ireland	204	51	51	1921				2.6	7.2	23.6	31.7	39.6					9.9	8.2	22.7	74	100	125	25
711	Belfast, N. E. Ireland	61		35	1915							34.5										100		25
712	Douglas, Isle of Man	277		46	1921				4.7	10.4	27.7	41.8	54.5					11.4	11.2	25.0	68	100	130	25
713	Stonyhurst, N. W. England	375	52	52	1921	1.25			2.5	12.7	30.6	46.5	60.5					8.2	5.4	27.3	66	100	130	25; 8
714	Seathwaite, N. W. England	422		35	1915							129.5										100		25
715	The Stye, N. W. England	1 070		35	1915							176.9										100		13
716	Glasgow, Scotland	180	51	51	1920				2.7	9.8	29.0	37.2	61.6					8.0	7.3	26.4	78	100	166	25
717	Ben Nevis, Scotland	4 405	20	20	1904				7.3	48.3	107.8	160.8	240.1					11.9	4.5	30.0	67	100	150	25
718	Port William, W. Scotland	171	46	46	1921				4.5	23.1	19.1	77.8	108.5					13.0	5.8	29.7	25	100	140	25
719	Stornoway, Hebrides Islands	51	49	49	1921				2.8	14.9	32.2	49.8	71.6					12.6	5.6	30.0	65	100	144	25
720	Deerness, Orkney Islands	160	49	49	1921				3.6	8.2	29.1	35.4	44.6					11.1	10.2	23.2	82	100	128	25
721	Inverness, N. Scotland	98		35	1915							26.9										100		25
722	Aberdeen, Scotland	45	36	50	1920				2.8	9.4	23.5	30.4	43.8					10.7	9.2	31.0	77	100	144	4
723	Dundee, Scotland	160		35	1915							27.4										100		25
724	Crieff, Scotland	432		35	1915							40.9										100		25
725	Edinburgh (Leith), Scotland	30	50	50	1920				2.8	7.2	15.9	23.3	37.5					10.3	12.0	30.9	68	100	161	25
726	Wolfelee, Cheviot Hills, Scotland	537		35	1915							37.6										100		25
727	Tynemouth, N. E. England	96	51	51	1921				3.2	7.8	18.6	24.5	40.9					12.3	13.0	31.8	76	100	167	25
738	Liverpool, W. England	188		35	1915							27.9										100		25
729	Derby, Central England	987	47	47	1921				2.8	10.3	32.4	48.4	65.8					10.2	5.8	21.3	67	100	135	25
730	Yarmouth, E. England	17	51	51	1921				4.8	8.6	19.4	24.5	44.2					12.0	19.6	35.0	79	100	180	25
731	Vlieland, N. Holland		26	26	1910				2.1	7.5	10.4	22.8	33.4					14.4	9.2	33.0	46	100	147	27
732	Leeghwater, Holland		45	45	1911				2.3	10.2	21.8	32.1	46.6					13.0	7.2	31.8	68	100	145	27
733	Utrecht-De Bilt, Holland	15	62	72	1920				2.9	8.5	17.7	28.7	41.1					10.1	10.2	29.7	63	100	144	4; 27
734	Ostend, Belgium	12		50	1900							25.7										100		9
735	Brussels, Belgium	187		50	1900							28.8										100		9
736	Bouillon, Belgium	774		50	1900							47.4										100		9
737	Treves, W. Germany	460		101	1924					9.4	15.1	26.9	39.2					8.3		34.9	56	100	146	4; 9
738	Freiburg, S. W. Germany	923		50	1900							33.4										100		9
739	Frankfort-am-Main, Germany	335		88	1924					8.4	14.1	24.3	36.8					10.9		34.6	58	100	151	4
740	Gutersloh, Germany	249		87	1924					8.2	19.1	30.0	38.4					9.3		27.3	64	100	128	4
741	Bremen, Germany			37								27.9										100		20
742	Berlin, Germany	115		74	1924					9.0	14.2	22.7	30.0					13.1		39.6	68	100	132	4
743	Dresden, Germany		27	58	1888				4.0	8.2	11.7	23.0	34.1					13.6	17.5	35.8	51	100	148	15
744	Breslau, Germany	482		66	1924					9.0	15.9	22.6	32.6					12.6		39.8	70	100	145	4
745	Prague, Czecho-Slovakia			50								18.5										100		20
746	Krynica, Poland	1 923		44	1920					12.0	21.2	33.4	48.4					11.1		35.9	63	100	145	4
747	Lemberg, Poland	978		45	1920					10.6	20.3	27.0	37.1					15.0		39.2	75	100	138	4
748	Warsaw, Poland	436	40	109	1920				3.4	6.8	14.6	21.2	46.5	0.8				14.9	16.2	32.3	69	100	220	20; 4
749	Konigsberg, E. Prussia	10		84	1924					9.8	12.9	20.5	33.3					13.3		48.0	63	100	163	4
750	Vilna, E. Poland	486		43	1915					8.1	10.6	20.4	28.7					15.7		39.6	54	100	141	4; 20
751	Riga, Latvia	66	30	31	1882				1.7	7.4	12.5	20.0	30.4	0.7				10.5	8.5	37.0	62	100	152	20
752	Tallinn, Esthonia	46	27	40	1891				2.0	7.5	7.1	19.2	27.7					14.3	10.4	39.1	37	100	144	20; 21
753	Leningrad, Russia	16	44	95	1920				2.3	7.3	12.1	19.2	29.3	0.6				14.7	12.0	38.0	63	100	153	20; 4
754	Moscow, Russia	538	27	64	1916				1.7	8.0	13.7	22.7	32.8	0.6				9.6	7.5	35.3	61	100	144	20; 4
755	Gorki, Russia	679	32	42	1891				3.1	8.1	12.8	19.9	38.5+	0.6				13.7	15.6	40.7	64	100	194+	20; 21
756	Kiev, Russia	600	23	62	1915				4.1	8.8	13.0	22.2	33.5	0.8				14.1	18.5	39.7	59	100	151	20; 4
757	Odessa, Russia	213	20	69	1915				3.1	6.6	9.0	15.5	24.9	1.3				14.1	19.9	42.4	58	100	160	20; 4; 22
758	Taganrog, Russia	115		30	1908				5.5	6.8	9.4	16.1	26.4					12.8	34.1	42.2	56	100	164	22; 20
759	Sevastopol, Crimea	132 76	16 30	34 30	1882 1908				2.1 4.6	4.7	8.4	15.2 14.1	25.3	1.0				8.2 7.8	13.8 32.5	30.8	55	100 100	186	20 22
760	Novorossisk, Russia	122	34	40	1914				4.0	10.2	11.7	27.1	45.5					12.8	14.6	37.6	43	100	168	4; 22; 20
761	Batum, Trans-Caucasia	10		26	1908				10.3	20.5	66.2	98.3	111.0					9.5	10.5	20.9	68	100	113	22; 20
762	Tiflis, Trans-Caucasia	1 335		70	1914					9.0	11.3	19.6	30.6	1.0				15.9		45.9	56	100	156	20; 4
763	Lenkoran, Trans-Caucasia	—66		27	1891					19.6	29.3	46.5	66.5	1.0				17.2		42.2	63	100	143	20; 21
764	Baku, Trans-Caucasia	7	20	39	1888				4.0	6.6	5.6	9.7	19.2	1.6				13.7	41.3	68.1	58	100	198	20; 21
765	Astrakhan, Russia	—46	32	68	1915				2.2	6.0	1.0	6.3	12.4	2.2				8.3	35.0	95.5	16	100	196	20; 4
766	Lugan, S. Russia	164	43	54	1891				2.5	5.7	8.8	14.9	24.5	0.9				12.7	16.8	48.2	59	100	164	20; 21
767	Nikolaewskoe, Russia	633		36	1915					6.0	10.2	14.5	22.6	1.2				14.5		41.3	70	100	155	4; 20
768	Kasan, E. Russia	266		54	1916					5.9	9.9	16.4	24.7					14.8		36.0	60	100	151	4; 20
769	Slatousk, E. Russia	1 476	44	55	1891				3.5	10.2	8.6	18.6	31.4	0.6				15.8	18.8	54.8	46	100	168	20; 21
770	Orenburg, E. Russia	374	32	64	1916				1.7	6.5	9.2	15.0	24.0	0.9				12.5	11.3	43.3	61	100	160	20; 4
771	Irgis, W. Siberia	367	18	26	1891				1.2	3.5	2.8	6.9	9.6	2.0				9.9	17.4	60.8	41	100	139	20; 21
772	Khiva, Turkestan	328		17	1891					2.3	0.8	2.1	3.4					26.6		109.4	38	100	162	20; 21
773	Tashkend, Turkestan	3 378		20	1891					5.3	9.4	13.1	18.0	1.8				19.0		40.4	72	100	137	20; 21
774	Barnaul, S. Siberia	479	43	54	1891				2.2	5.8	4.2	11.5	17.7	1.2				14.3	19.1	50.4	37	100	154	20; 21
775	Irkutsk, S. Siberia	1 509		17	1891					7.1	9.6	14.5	18.8	1.6				15.8		49.0	66	100	130	20; 21
776	Nertschinsk, S. Siberia	2 155	42	51	1891				6.2	11.2	7.2	16.0	25.9	1.2				26.9	38.7	69.8	45	100	162	20; 21
777	Nikolaewsk-am-Amur, S. Siberia	108		60	1919				2.5	9.0	7.8	17.6	30.2+	1.2				15.2	14.2	51.0	44	100	172+	17
778	Ajan, E. Siberia	33	12	12	1912				3.6	19.1	16.2	35.0	83.0	1.3				22.2	10.3	54.5	46	100	237	17
779	Okhotsk, E. Siberia	20	23	23	1919				1.9	11.2	6.4	11.3	16.5					20.6	16.9	100.0	57	100	147	17
780	Gishiga, E. Siberia	40		24	1919					3.9	5.8	9.2	12.2+					18.0		42.5	63	100	133	17
781	Petropavlovsk, E. Siberia (S. Kamchatka)	52		8	1891					16.1	36.4	47.0	76.0					12.4		34.3	78	100	162	20; 21
782	Novo-Mariinski, N. E. Siberia	16		21	1919					4.3	4.7	7.4	10.3					20.6		58.1	64	100	139	17
783	Markovo, N. E. Siberia	66	25	25	1919				1.6	6.1	5.4	7.9	10.0					24.1	20.3	77.2	68	100	127	17
784	Verkhoyansk, N. Siberia	328	34	34	1919				0.8	3.4	2.0	5.0	9.1					17.6	16.0	68.0	40	100	182	17
785	Yakutsk, Siberia	354	36	36	1923				1.4	4.6	4.0	7.4	12.8	0.7				14.5	18.9	62.2	54	100	173	17
786	Viluisk, N. Siberia	394	26	26	1923				1.8	4.8	5.7	8.9	11.5					18.0	20.3	54.0	64	100	129	17
787	Turuchansk, N. W. Siberia	131	39	39	1922				1.4	5.7	5.0	13.2	20.8	0.7				14.9	10.6	43.2	38	100	158	17
788	Obdorsk, N. W. Siberia	59		27	1909					5.8	4.5	10.1	14.4	1.1				20.0		57.3	44	100	142	17
789	Beresov, N. W. Siberia	134	28	28	1909				2.6	6.8	5.6	13.8	21.9					16.5	18.8	49.3	41	100	158	17
790	Surgut, N. W. Siberia	138	24	24	1909				2.7	6.0	11.8	15.8	23.0					16.4	17.0	37.8	74	100	146	17
791	Bogoslovsk, W. Siberia	623	42	64	1891				2.6	7.7	5.4	16.4	28.4	0.9				17.4	15.8	47.0	33	100	173	20; 21
792	Ekaterinburg, W. Siberia	921	45	80	1915				3.7	8.4	6.9	15.2	24.4	1.0				19.0	24.3	56.4	45	100	160	20; 4
793	Perm, E. Russia	534		33	1915					7.2	17.6	23.2	31.4	0.7				12.3		31.0	76	100	135	4
794	Ust-Zylma, N. E. Russia	82		21	1916					4.6	12.2	16.6	21.9					15.9		27.7	73	100	132	4
795	Archangel, N. Russia	22		51	1916					5.1	9.5	16.8	29.8	1.0				14.3		30.2	56	100	177	4; 20
796	Kem, N. W. Russia	20	18	28	1891				1.8	4.8	10.2	14.5	21.7					13.6	12.4	33.0	70	100	150	20; 21
797	Helsingfors, Finland	38		80	1924					7.6	14.3	24.4	32.9					9.2		31.1	59	100	139	4
798	Abo, S. W. Finland	66		49	1800					7.3	15.7	23.8	33.0					12.5		30.6	66	100	138	20
799	Upsala, Sweden	79		74	1924					7.9	12.3	21.2	32.0					12.6		37.4	58	100	151	4
800	Ostersund, Sweden	1 017		51	1924					6.3	11.2	18.6	26.4					16.4		33.9	60	100	142	4
801	Haparanda, Sweden	30		65	1924					7.4	9.2	19.2	30.1					11.6		38.6	48	100	157	4
802	Karesuanda, Sweden	1 092		46	1924					6.1	6.8	12.4	20.2					18.9		49.1	55	100	163	4
803	Kola, Lapland	33		14	1891					2.4	4.9	6.9	10.6					15.5		34.8	71	100	154	21; 20
804	Gjesvar, N. Norway	28		37	1920					10.4	17.3	29.1	46.1					11.4		35.6	59	100	158	4
805	Bodo, N. Norway	75		53	1920					12.4	20.1	35.6	58.0					9.4		34.8	56	100	162	4
806	Christiania, S. Norway	85		58	1920					8.1	14.5	22.1	31.7					10.6		36.6	65	100	143	4
807	Copenhagen, Denmark	16		100	1920					8.0	10.4	22.5	30.3					7.9		35.5	46	100	134	4
808	Lerwick, Shetland Islands	112	50	50	1921				2.8	10.8	27.6	38.0	57.3					11.2	7.8	28.4	72	100	150	25
809	Thorshavn, Faroes Islands	89		48	1920					13.7	37.6	57.2	77.1					11.5		33.9	66	100	134	4
810	Berufjord, E. Iceland	64		51	1923					15.1	22.5	45.9	68.4					10.8		32.9	49	100	149	4
811	Vestmanno, S. W. Iceland	27		43	1923					12.9	37.3	53.7	70.7					10.8		25.8	69	100	131	4
812	Stykkisholm, N. W. Iceland	58		67	1923					8.6	13.8	25.7	40.5					10.0		33.4	54	100	157	4
813	Grimsey, N. Iceland	78		41	1923					7.3	2.6	11.7	27.1					11.6		62.0	22	100	231	4
814	Angmagsalik, E. Greenland	109		22	1920					18.6	21.8	35.6	58.4					17.5		52.2	61	100	164	4
815	Upernivik, N. W. Greenland	68		47	1920					5.3	3.4	9.0	17.8					10.1		59.0	38	100	198	4
816	Jacobshavn, W. Greenland	48		47	1920					5.9	5.2	9.1	17.8					15.2		65.0	57	100	196	4
817	Godthaab, S. W. Greenland	35		37	1920					11.0	12.2	23.7	35.5					13.1		46.5	51	100	150	4
818	Ivigtut, S. Greenland	21		45	1920					20.6	20.9	44.8	72.5					9.8		46.0	47	100	161	4
819	Horta, Azores	210		22	1923					9.9	30.5	43.0	64.0					11.2		22.0	71	100	149	4
820	Ponta-Delgada, Azores	123		30	1923					9.5	19.6	28.3	42.2					5.2		33.5	69	100	149	4

(Continued.)

Station No.	January. Mean month.	January. Maximum. 24 hours.	January. Maximum. Month.	February. Mean month.	February. Maximum. 24 hours.	February. Maximum. Month.	March. Mean month.	March. Maximum. 24 hours.	March. Maximum. Month.	April. Mean month.	April. Maximum. 24 hours.	April. Maximum. Month.	May. Mean month.	May. Maximum. 24 hours.	May. Maximum. Month.	June. Mean month.	June. Maximum. 24 hours.	June. Maximum. Month.	July. Mean month.	July. Maximum. 24 hours.	July. Maximum. Month.	August. Mean month.	August. Maximum. 24 hours.	August. Maximum. Month.	September. Mean month.	September. Maximum. 24 hours.	September. Maximum. Month.	October. Mean month.	October. Maximum. 24 hours.	October. Maximum. Month.	November. Mean month.	November. Maximum. 24 hours.	November. Maximum. Month.	December. Mean month.	December. Maximum. 24 hours.	December. Maximum. Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
703	2.79	1.6	5.7	1.89	1.6	3.9	1.94	1.1	3.7	1.90	1.6	4.7	2.05	2.0	5.5	1.95	1.8	6.2	2.56	2.0	6.1	3.04	3.4	7.6	1.92	2.1	5.1	2.69	2.9	7.4	2.67	2.6	6.5	2.45	1.6	7.6
704	4.15	...	...	3.70	...	...	3.01	...	...	2.68	...	...	2.43	...	...	2.67	...	...	2.88	...	...	3.78	...	...	2.97	...	...	4.09	...	...	4.20	...	...	5.32	...	...
705	6.67	2.8	10.6	5.07	1.9	11.0	4.38	1.7	10.8	3.76	1.6	6.6	3.19	1.5	6.6	3.47	1.7	7.8	3.92	1.9	8.7	4.83	2.3	8.5	4.49	2.5	10.0	5.87	2.5	10.4	5.44	2.7	9.7	6.50	1.9	11.4
706	5.92	...	...	5.22	...	...	4.71	...	...	3.31	...	...	3.06	...	...	2.91	...	...	3.32	...	...	4.42	...	...	3.58	...	...	5.37	...	...	5.61	...	...	7.28	...	...
707	3.20	...	...	2.54	...	...	2.29	...	...	2.18	...	...	3.21	...	...	2.42	...	...	2.82	...	...	3.48	...	...	2.31	...	...	3.15	...	...	3.06	...	...	3.46	...	...
708	3.21	...	...	2.78	...	...	2.70	...	...	2.37	...	...	2.45	...	...	2.60	...	...	3.18	...	...	4.17	...	...	2.67	...	...	3.12	...	...	2.41	...	...	3.68	...	...
709	5.26	...	...	4.46	...	...	4.52	...	...	3.14	...	...	3.75	...	...	3.00	...	...	3.64	...	...	4.51	...	...	2.92	...	...	3.18	...	...	2.95	...	...	3.55	...	...
710	2.52	1.1	5.5	2.22	1.2	4.5	2.35	1.1	4.4	2.10	1.3	4.0	2.38	1.6	6.2	2.52	2.2	5.6	2.89	2.1	7.1	3.62	2.6	7.0	2.46	1.7	5.4	2.72	1.6	5.4	2.84	1.7	6.5	3.13	1.6	7.2
711	2.56	...	...	2.47	...	...	2.55	...	...	2.22	...	...	2.31	...	...	2.48	...	...	2.97	...	...	3.69	...	...	2.56	...	...	3.29	...	...	3.37	...	...	3.65	...	...
712	3.39	2.0	7.0	3.27	1.3	7.2	2.97	1.7	5.5	2.47	1.2	5.0	2.53	2.6	6.6	2.47	1.6	4.9	3.06	1.8	6.6	3.89	2.2	7.7	3.30	4.7	9.7	4.62	2.2	9.2	4.79	3.6	10.4	5.01	2.0	8.3
713	4.28	2.1	8.6	3.35	2.0	6.8	3.69	1.5	7.1	2.71	1.2	5.7	2.85	1.6	6.5	3.07	1.7	8.7	3.87	2.5	8.3	5.00	1.9	9.8	3.82	2.0	12.7	4.49	2.5	10.8	4.51	3.2	8.3	4.85	1.8	10.6
714	13.78	...	...	11.87	...	...	11.15	...	...	7.42	...	...	7.37	...	...	6.52	...	...	8.46	...	...	11.57	...	...	9.92	...	...	12.00	...	...	13.58	...	...	16.34	...	...
715	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
716	3.27	1.5	7.4	2.88	2.7	9.2	2.67	1.1	6.8	2.08	1.2	4.4	2.59	1.1	4.7	2.52	2.4	9.1	3.09	2.1	6.5	3.91	2.6	6.9	2.98	1.6	9.9	3.40	2.4	8.3	3.64	1.8	6.5	4.15	1.8	7.4
717	18.40	6.8	35.3	13.55	6.7	36.2	15.24	5.5	37.9	8.46	4.0	20.2	7.91	5.9	15.8	7.53	2.7	14.6	10.79	3.2	15.2	13.35	4.0	21.0	15.75	5.4	43.5	15.43	7.3	37.3	15.35	5.4	32.5	19.06	5.4	48.3
718	9.58	3.7	20.0	7.42	3.2	19.1	6.60	2.9	15.9	4.43	2.6	9.7	3.86	1.7	6.7	3.50	1.8	11.3	4.85	3.3	11.6	6.14	2.4	12.0	6.28	2.8	14.0	7.03	4.5	14.1	8.07	3.8	15.9	10.06	3.7	23.1
719	5.17	2.6	9.0	4.46	1.6	8.0	4.10	1.5	8.1	3.03	1.2	6.9	2.56	1.9	5.2	2.32	1.1	5.4	3.03	1.7	6.8	3.97	2.2	8.5	3.96	1.9	8.5	5.18	2.1	9.8	5.33	1.9	9.8	6.25	1.6	14.9
720	3.43	1.2	5.9	3.01	1.7	5.9	2.81	1.3	5.8	2.07	1.1	4.3	1.99	1.8	3.8	1.84	1.0	3.7	2.57	1.6	5.2	2.87	3.6	6.9	2.90	1.6	6.6	3.79	1.7	7.1	3.93	1.6	8.2	4.19	1.7	7.6
721	2.48	...	...	2.18	...	...	3.13	...	...	1.59	...	...	1.81	...	...	1.87	...	...	2.55	...	...	2.49	...	...	2.29	...	...	2.38	...	...	2.50	...	...	3.06	...	...
722	2.13	1.4	4.1	2.10	1.5	4.7	2.36	1.4	5.2	1.99	1.5	4.8	2.26	1.1	4.9	1.86	1.8	4.6	2.91	1.5	5.9	2.94	1.9	7.0	2.48	2.5	6.2	3.01	1.9	6.2	3.17	2.8	6.2	3.24	1.5	9.4
723	1.95	...	...	1.88	...	...	2.06	...	...	1.70	...	...	2.09	...	...	1.80	...	...	2.74	...	...	3.38	...	...	2.08	...	...	2.66	...	...	2.44	...	...	2.86	...	...
724	4.09	...	...	3.52	...	...	3.20	...	...	2.19	...	...	2.49	...	...	2.64	...	...	2.97	...	...	4.21	...	...	2.88	...	...	3.93	...	...	4.94	...	...	4.48	...	...
725	1.99	1.4	4.5	1.42	1.6	5.3	1.75	1.6	4.4	1.29	1.2	4.3	1.83	1.4	5.3	1.73	1.4	4.7	2.55	2.7	5.6	2.94	2.8	6.0	1.76	1.4	5.0	2.41	2.6	7.2	1.95	1.8	5.2	2.00	1.0	5.7
726	3.20	...	...	2.26	...	...	3.35	...	...	2.26	...	...	2.34	...	...	2.84	...	...	3.06	...	...	3.34	...	...	2.57	...	...	3.84	...	...	3.87	...	...	4.18	...	...
727	1.62	0.9	4.2	1.27	2.0	4.3	1.81	1.4	4.2	1.42	1.3	3.9	1.99	2.5	5.9	1.98	2.6	5.4	2.40	1.7	5.4	2.75	2.4	6.1	1.80	1.6	5.0	3.01	3.2	7.8	2.13	2.4	5.8	2.18	1.8	5.8
728	2.12	...	...	1.68	...	...	1.90	...	...	1.63	...	...	1.90	...	...	2.20	...	...	2.59	...	...	3.06	...	...	2.41	...	...	3.27	...	...	2.50	...	...	2.63	...	...
729	4.47	2.2	8.8	3.75	2.0	9.1	4.13	1.7	7.5	2.94	1.9	8.1	3.10	1.5	6.1	3.22	2.8	8.1	3.93	2.0	7.9	4.38	1.8	8.8	3.24	1.6	10.2	4.91	2.4	10.3	4.68	1.6	9.6	5.67	1.8	10.0
730	1.74	1.1	3.7	1.49	0.8	3.4	1.73	0.9	3.7	1.49	1.3	3.9	1.73	1.2	3.7	1.77	1.7	4.3	2.34	1.9	5.9	2.47	4.8	8.2	1.96	2.8	5.9	2.93	2.2	8.6	2.41	3.5	8.2	3.44	2.5	6.9
731	1.88	0.6	4.0	1.34	0.7	3.8	1.50	0.7	3.1	1.24	2.0	3.0	1.42	1.4	3.7	1.46	1.6	3.4	1.72	1.4	3.6	2.76	2.1	6.3	2.29	1.2	7.1	3.27	1.1	7.5	2.29	1.2	6.9	2.01	0.9	4.9
732	2.18	0.9	6.3	1.74	1.4	4.0	1.96	1.1	4.4	1.82	2.3	7.2	1.98	1.1	4.4	2.49	1.9	5.2	2.88	1.8	6.0	1.80	2.0	9.7	3.21	1.7	7.3	4.19	1.8	10.3	2.91	1.0	7.8	2.99	1.8	4.8
733	2.14	1.2	4.0	1.72	1.4	4.8	1.97	1.0	4.1	1.71	1.3	4.1	1.94	1.4	5.2	2.34	1.7	5.8	3.01	1.6	3.7	3.32	2.9	7.2	2.58	1.4	5.6	2.89	1.5	8.6	2.36	1.6	4.7	2.71	1.7	6.0
734	1.90	...	...	1.38	...	...	1.70	...	...	1.42	...	...	1.90	...	...	2.05	...	...	2.33	...	...	2.79	...	...	2.56	...	...	3.00	...	...	2.53	...	...	2.17	...	...
735	2.17	...	...	1.78	...	...	1.90	...	...	1.82	...	...	2.33	...	...	2.60	...	...	2.95	...	...	2.91	...	...	2.64	...	...	2.84	...	...	2.40	...	...	2.48	...	...
736	4.35	...	...	2.94	...	...	3.55	...	...	2.72	...	...	3.51	...	...	3.53	...	...	4.03	...	...	3.77	...	...	3.60	...	...	4.94	...	...	4.82	...	...	3.40	...	...
737	1.94	...	5.0	1.67	...	4.8	1.89	...	4.9	1.74	...	5.1	2.27	...	5.6	2.63	...	6.1	2.79	...	6.5	2.64	...	6.5	2.24	...	9.4	2.44	...	7.7	2.33	...	7.1	2.43	...	6.1
738	1.65	...	...	1.48	...	...	2.05	...	...	2.72	...	...	3.46	...	...	4.13	...	...	4.02	...	...	3.35	...	...	3.11	...	...	3.02	...	...	2.32	...	...	2.03	...	...
739	1.76	...	4.8	1.37	...	4.0	1.59	...	4.3	1.44	...	5.7	2.06	...	6.1	2.48	...	7.7	2.79	...	8.2	2.64	...	8.4	1.96	...	4.2	2.15	...	6.7	1.98	...	6.6	2.05	...	4.6
740	2.34	...	4.9	2.01	...	5.2	2.05	...	5.1	1.75	...	5.8	2.24	...	5.8	2.79	...	8.2	3.26	...	7.4	2.93	...	6.8	2.19	...	6.1	2.37	...	6.6	2.29	...	6.3	2.61	...	7.4
741	...	...	...	...	...	...	...	...	...	1.61	...	...	...	...	...	...	...	...	3.38	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
742	1.67	...	3.9	1.41	...	4.9	1.60	...	5.3	1.52	...	4.2	1.91	...	5.7	2.35	...	5.6	2.97	...	9.0	2.29	...	6.6	1.67	...	3.8	2.74	...	5.1	1.65	...	4.8	1.83	...	4.5
743	1.21	0.7	3.9	1.22	0.7	3.7	1.44	0.8	4.4	1.58	1.3	5.3	2.27	1.3	6.1	3.10	2.9	7.8	3.11	4.0	8.2	2.46	1.8	7.7	1.87	1.5	6.0	1.49	2.2	4.5	1.62	1.8	3.5	1.54	1.2	4.3
744	1.27	...	2.9	1.15	...	3.5	1.46	...	3.3	1.55	...	3.9	2.29	...	6.0	2.40	...	4.6	3.21	...	7.2	2.85	...	9.0	1.87	...	5.2	1.83	...	3.7	1.46	...	4.4	1.49	...	3.3
745	...	...	...	0.83	...	...	...	...	...	...	...	...	...	...	...	2.83	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
746	2.36	...	7.6	1.77	...	4.1	2.05	...	4.2	2.28	...	6.3	3.23	...	6.1	4.06	...	10.5	4.69	...	10.8	3.70	...	12.0	2.99	...	7.6	2.44	...	6.2	1.73	...	3.9	2.13	...	4.4
747	1.26	...	3.1	1.30	...	3.6	1.69	...	4.3	1.81	...	5.0	2.68	...	6.8	3.58	...	7.2	4.06	...	10.6	3.23	...	8.2	2.17	...	7.6	2.06	...	5.0	1.65	...	4.2	1.57	...	4.0
748	1.21	0.8	3.0	1.08	0.6	2.1	1.35	0.9	3.2	1.49	1.3	3.9	1.89	1.5	4.6	2.57	2.0	4.3	3.18	3.6	6.8	2.96	2.4	5.5	1.90	1.8	3.5	1.67	1.4	5.4	1.47	1.1	3.4	1.40	0.8	2.9
749	1.27	...	4.3	1.05	...	3.4	1.04	...	3.3	1.07	...	3.6	1.46	...	5.0	1.85	...	5.6	2.36	...	7.9	2.72	...	9.8	2.40	...	7.1	1.91	...	7.0	1.80	...	5.3	1.59	...	5.9
750	1.02	...	3.0	0.88	...	2.6	0.99	...	2.2	1.15	...	2.8	1.93	...	6.4	2.56	...	5.6	2.86	...	5.6	3.20	...	8.1	1.89	...	3.9	1.53	...	3.3	1.31	...	3.1	1.16	...	3.6
751	1.20	0.6	2.8	0.81	0.7	2.0	1.00	0.7	2.3	1.09	0.6	2.5	1.68	1.1	4.2	3.10	1.5	7.4	2.33	1.1	6.5	2.49	1.7	5.6	2.19	1.1	5.8	1.96	1.3	5.9	1.90	1.0	4.1	1.36	0.7	3.1
752	1.01	0.7	2.1	0.81	0.6	1.9	0.95	0.7	3.5	0.95	0.6	2.5	1.64	1.5	3.0	1.60	1.3	4.0	2.00	1.9	4.5	2.75	3.0	7.5	2.18	2.0	6.6	2.14	1.4	5.5	1.84	1.2	4.1	1.30	0.7	5.2
753	0.94	0.6	2.6	0.89	0.8	2.3	0.89	1.1	2.3	1.03	0.8	3.2	1.64	1.2	4.4	1.99	2.0	5.7	2.47	2.2	5.4	2.82	2.3	7.3	2.14	1.8	7.0	1.76	1.0	4.1	1.40	0.7	3.5	1.23	1.1	3.7
754	1.30	0.7	3.4	1.12	0.7	2.6	1.26	0.6	3.4	1.41	0.8	4.2	1.90	1.1	4.0	2.34	1.5	7.5	3.06	1.4	6.5	2.90	1.7	5.4	2.17	1.3	8.0	1.84	1.0	5.7	1.70	1.1	4.1	1.61	1.0	4.5
755	0.82	0.6	2.8	0.82	0.7	3.1	1.01	1.3	2.7	1.35	1.9	6.3	1.98	2.1	6.2	2.73	1.9	8.1	3.10	1.8	7.8	2.27	3.1	4.7	1.74	1.6	5.6	1.60	1.8	5.8	1.28	1.0	3.8	1.07	0.7	2.9
756	1.28	1.3	3.5	1.07	1.2	3.3	1.58	0.9	4.0	1.72	1.7	4.6	1.85	1.5	5.7	2.64	2.6	6.1	3.13	3.5	8.8	2.31	4.1	6.2	1.83	2.0	4.3	1.77	1.9	5.6	1.48	1.8	4.6	1.55	1.3	4.0
757	1.00	0.8	4.0	0.82	0.7	3.7	1.07	1.0	2.8	1.00	1.4	4.0	1.22	1.0	4.8	2.20	3.1	6.6	1.92	3.0	5.1	1.26	1.9	5.9	1.27	1.6	5.7	1.22	2.1	4.2	1.22	1.1	4.8	1.20	0.7	5.3
758	1.12	1.1	4.0	1.10	1.1	4.1	1.12	0.8	3.2	1.23	1.0	2.5	1.35	1.7	6.1	2.19	2.1	4.2	2.06	5.5	6.8	1.03	1.4	2.3	1.08	1.6	4.5	1.11	1.2	1.9	1.25	1.8	8.2	1.46	1.1	4.5
759	1.09 1.17	0.8 0.8	3.1 ...	0.97 1.13	1.1 1.0	2.2 ...	0.99 1.80	0.7 1.1	3.5 ...	1.08 1.11	2.1 2.1	4.1 ...	0.78 0.74	0.8 0.9	2.0 ...	1.05 1.18	1.8 1.9	2.7 ...	1.25 1.10	2.1 4.6	4.7 ...	1.22 0.87	1.7 2.2	3.4 ...	1.86 1.17	1.2 1.4	6.2 ...	1.43 1.43	1.1 1.7	3.9 ...	1.72 1.47	2.0 1.8	6.4 ...	2.03 1.45	1.0 1.0	4.0 ...
760	3.48	2.6	10.2	2.44	2.3	8.5	2.43	3.9	8.0	1.84	2.2	4.6	1.61	2.6	6.0	2.28	3.0	6.4	2.03	2.4	7.8	1.35	2.9	7.8	2.08	2.6	5.6	1.68	1.4	4.4	2.67	4.0	8.5	2.32	2.2	9.4
761	10.11	3.6	12.4	7.14	2.9	11.6	5.89	2.9	13.5	4.93	2.9	6.7	3.26	3.9	3.9	6.72	6.2	7.4	6.42	5.8	8.5	9.35	10.3	20.6	12.10	7.8	17.4	9.23	4.9	11.4	12.73	6.1	11.2	10.38	3.9	14.5
762	0.58	...	2.6	0.77	...	2.1	1.12	...	3.6	2.12	...	7.4	3.12	...	9.0	3.75	...	7.2	1.96	...	5.3	1.51	...	7.8	2.00	...	7.8	1.44	...	4.6	1.27	...	4.2	0.80	...	3.3
763	4.59	...	8.6	3.16	...	6.1	3.46	...	7.4	2.48	...	5.5	1.47	...	4.0	1.01	...	3.6	1.00	...	6.0	1.75	...	4.8	8.15	...	17.6	8.00	...	19.6	6.94	...	14.2	4.35	...	9.1
764	1.24	4.0	6.6	0.87	2.1	3.5	0.88	1.8	4.6	0.82	1.7	4.1	0.62	2.4	3.5	0.30	0.8	1.4	0.21	0.7	3.0	0.27	0.7	1.2	0.75	1.2	3.4	1.19	1.3	4.0	1.17	1.5	3.9	1.25	1.3	3.1
765	0.82	0.9	6.0	0.38	1.2	2.4	0.46	2.2	3.1	0.51	0.6	2.2	0.70	2.1	2.7	0.72	1.6	4.2	0.52	0.8	1.9	0.49	1.1	2.3	0.57	1.1	2.8	0.46	0.9	1.7	0.44	1.0	3.0	0.54	0.7	3.1
766	0.79	0.6	2.2	0.63	1.1	1.8	0.87	0.6	3.2	1.14	1.2	3.1	1.69	2.1	4.8	2.07	2.5	5.6	1.90	1.8	5.7	1.40	1.9	5.7	1.08	1.1	3.3	1.18	1.0	4.0	1.21	1.1	3.5	0.98	0.6	3.4
767	0.88	...	2.2	0.67	...	1.9	0.78	...	2.3	0.85	...	1.6	1.38	...	4.6	2.11	...	6.0	1.12	...	3.3	1.63	...	4.5	1.15	...	3.0	1.49	...	3.5	1.15	...	2.2	1.05	...	2.4
768	0.72	...	1.6	0.58	...	1.7	0.72	...	2.3	0.99	...	3.1	1.50	...	4.3	2.30	...	4.8	2.43	...	5.9	2.14	...	6.6	1.65	...	4.2	1.22	...	4.1	1.18	...	4.3	0.88	...	3.2
769	0.88	0.4	2.0	0.56	0.7	2.5	0.61	0.7	2.8	0.84	0.6	3.0	1.74	1.1	4.5	2.90	3.5	6.5	3.46	2.0	7.5	2.82	2.4	10.2	1.84	1.2	6.2	1.41	1.1	4.0	1.00	0.6	2.2	0.73	0.5	2.4
770	1.16	0.9	3.7	0.83	0.9	4.8	0.94	1.3	4.8	0.89	1.2	3.3	1.42	1.7	4.6	1.87	1.5	6.5	1.57	1.3	4.4	1.29	1.3	4.3	1.11	1.7	4.6	1.22	1.5	3.6	1.33	0.8	3.8	1.46	0.8	2.8
771	0.54	1.0	2.2	0.36	0.7	1.1	0.48	0.7	1.5	0.68	1.2	3.5	0.84	0.9	2.4	0.85	0.9	3.0	0.62	0.8	1.4	0.41	0.8	1.5	0.50	0.7	1.2	0.57	0.8	2.3	0.43	0.6	1.1	0.56	0.8	1.5
772	0.25	...	0.6	0.37	...	1.2	0.56	...	2.3	0.49	...	2.1	0.17	...	0.7	0.12	...	1.0	0.02	...	0.1	0.04	...	0.3	0.04	...	0.2	0.17	...	0.6	0.13	...	0.5	0.19	...	1.0
773	1.74	...	3.7	1.54	...	3.6	2.50	...	5.3	2.14	...	4.3	0.92	...	2.6	0.31	...	0.8	0.06	...	0.4	0.06	...	0.4	0.20	...	0.7	0.88	...	2.5	0.93	...	3.0	1.77	...	10.3
774	0.48	0.4	1.0	0.37	0.3	0.7	0.39	0.5	1.0	0.49	0.6	1.3	1.18	1.1	2.5	1.48	1.2	4.0	1.90	2.0	5.3	1.63	2.2	3.8	0.96	1.0	3.1	0.99	1.1	2.9	0.85	0.8	2.4	0.74	0.5	2.1
775	0.94	...	1.9	0.45	...	0.9	0.35	...	1.2	0.54	...	2.0	1.16	...	1.4	2.30	...	7.1	2.87	...	6.4	2.45	...	3.5	1.60	...	3.5	0.69	...	2.6	0.59	...	2.0	0.83	...	2.3
776	0.07	0.2	0.5	0.07	0.2	0.2	0.20	0.9	1.0	0.54	1.4	1.6	1.10	1.3	3.6	2.51	1.6	7.0	4.30	3.2	9.3	4.32	6.2	11.2	1.96	2.2	4.4	0.55	1.2	1.3	0.28	0.6	0.8	0.15	0.3	0.4
777	0.50	0.6	2.9	0.48	0.9	2.6	0.67	0.3	3.0	1.15	0.8	4.8	1.31	0.7	3.6	1.55	0.9	4.2	2.06	1.4	7.2	3.07	1.6	8.6	2.67	1.2	9.0	1.99	2.5	8.4	1.24	1.1	4.5	0.90	2.4	3.7
778	0.30	0.2	1.1	0.29	0.3	1.1	0.45	0.2	2.9	0.67	0.5	3.1	1.68	1.5	3.9	4.79	2.9	12.8	4.90	3.6	16.4	7.78	3.1	19.1	6.40	2.1	12.8	4.65	1.6	16.0	2.26	0.6	10.1	0.80	0.4	5.2
779	0.08	0.2	0.3	0.09	0.1	0.3	0.17	0.2	0.4	0.35	0.2	1.6	1.60	0.9	4.2	1.69	1.3	4.6	2.31	1.9	11.2	2.42	1.2	6.8	2.10	0.8	5.6	0.96	0.4	2.6	0.15	0.2	0.5	0.11	0.2	0.4
780	0.33	...	1.6	0.28	...	1.0	0.44	...	1.2	0.31	...	0.9	0.36	...	1.2	0.74	...	1.5	1.66	...	3.9	1.76	...	3.4	1.24	...	3.0	1.04	...	1.8	0.63	...	1.8	0.40	...	1.6
781	4.03	...	8.5	3.30	...	5.7	5.86	...	16.1	3.42	...	10.5	3.62	...	9.7	1.84	...	3.2	2.74	...	4.5	2.02	...	3.1	4.91	...	11.4	4.20	...	9.7	6.87	...	15.4	5.20	...	10.5
782	0.22	...	0.9	0.26	...	0.9	0.24	...	0.7	0.19	...	0.8	0.30	...	0.9	0.95	...	2.4	1.29	...	3.1	1.52	...	4.3	1.13	...	2.8	0.53	...	1.7	0.25	...	0.8	0.39	...	1.1
783	0.32	0.2	1.6	0.28	0.2	0.7	0.29	0.3	0.8	0.15	0.1	0.5	0.32	0.3	0.7	0.77	0.8	2.2	1.47	0.8	2.9	1.91	1.6	6.1	1.07	1.4	3.9	0.48	0.3	1.0	0.46	0.8	1.5	0.37	0.8	0.8
784	0.16	0.1	0.6	0.13	0.1	0.4	0.10	0.1	0.3	0.16	0.2	0.7	0.26	0.5	0.6	0.88	0.8	3.4	1.01	0.8	2.9	1.00	0.4	2.4	0.51	0.5	1.7	0.33	0.4	1.5	0.26	0.3	0.8	0.15	0.1	0.4
785	0.24	0.2	0.8	0.20	0.2	0.7	0.13	0.1	0.5	0.25	0.5	0.8	0.50	0.3	1.3	1.07	1.4	4.6	1.32	0.9	4.0	1.64	0.7	4.3	0.39	0.6	1.8	0.47	0.4	1.3	0.38	0.2	1.0	0.28	0.3	0.7
786	0.34	0.2	0.8	0.30	0.2	0.9	0.28	0.2	0.6	0.27	0.2	0.7	0.89	0.8	2.1	1.23	0.6	3.2	1.38	0.6	2.7	1.60	1.8	4.8	1.07	0.7	2.8	0.65	0.2	1.1	0.83	0.2	0.9	0.39	0.2	0.7
787	0.47	0.2	1.7	0.37	0.1	0.9	0.43	0.1	2.1	0.57	0.3	1.9	1.04	0.4	3.1	1.61	1.4	3.8	1.97	1.0	5.7	2.27	0.9	5.4	2.02	1.0	4.0	1.15	0.6	3.1	0.77	0.4	2.0	0.55	0.1	1.2
788	0.29	...	0.6	0.30	...	0.7	0.24	...	0.6	0.25	...	0.9	0.67	...	1.9	1.85	...	3.1	1.96	...	5.2	2.02	...	5.8	1.46	...	3.0	0.70	...	1.7	0.47	...	1.0	0.29	...	1.0
789	0.63	0.3	1.6	0.38	0.2	1.0	0.51	0.7	1.9	0.60	0.4	2.3	1.32	1.5	5.2	2.01	2.1	5.4	2.29	1.6	6.8	2.31	2.6	4.8	1.58	1.5	3.6	0.95	0.6	2.9	0.72	0.5	2.2	0.53	0.5	1.2
790	0.56	0.3	1.3	0.33	0.2	0.7	0.42	0.3	1.2	0.44	0.3	1.1	1.42	1.6	4.4	2.47	1.4	5.4	2.60	2.7	6.0	2.96	1.8	5.8	1.95	1.1	4.1	1.23	0.6	2.8	0.76	0.6	1.4	0.68	0.3	1.3
791	0.52	0.6	1.3	0.55	0.6	2.0	0.62	0.8	1.7	0.86	0.6	2.9	1.67	1.4	4.5	2.18	1.5	6.0	2.88	2.9	7.7	2.77	1.5	7.0	1.74	1.8	4.5	1.18	0.7	3.5	0.82	0.7	2.1	0.65	0.7	1.9
792	0.40	0.4	1.9	0.32	0.4	1.1	0.37	0.4	1.4	0.61	0.6	2.6	1.66	1.8	5.1	2.78	1.4	6.3	2.80	3.7	8.4	2.53	3.3	7.8	1.42	2.7	4.8	0.91	0.5	3.0	0.74	0.7	2.2	0.56	0.4	3.2
793	1.45	...	2.4	1.17	...	2.3	1.15	...	3.2	1.00	...	3.5	2.00	...	4.0	2.73	...	5.5	2.78	...	6.2	2.87	...	7.2	2.32	...	4.7	1.89	...	3.4	3.06	...	4.4	1.70	...	4.0
794	0.78	...	1.6	0.66	...	1.2	0.65	...	1.5	0.63	...	1.7	1.31	...	2.4	2.00	...	3.4	2.64	...	4.6	2.37	...	4.8	2.11	...	4.5	1.34	...	2.0	1.06	...	1.8	0.83	...	2.6
795	0.90	...	2.2	0.74	...	1.6	0.85	...	2.8	0.73	...	1.3	1.17	...	3.3	1.74	...	4.7	2.38	...	4.9	2.41	...	5.1	2.10	...	4.2	1.63	...	3.5	1.19	...	3.0	0.90	...	2.2
796	0.63	0.6	1.3	0.43	0.3	0.9	0.54	0.5	1.1	0.66	0.7	1.7	1.39	0.9	3.4	1.62	1.4	3.7	2.28	1.0	4.7	1.94	1.8	4.1	1.97	1.2	4.8	1.32	0.7	2.3	0.94	0.6	2.2	0.79	0.8	1.8
797	1.77	...	4.6	1.46	...	3.6	1.38	...	4.4	1.42	...	3.9	1.77	...	3.9	1.81	...	4.4	2.34	...	7.6	2.90	...	6.5	2.52	...	5.9	2.60	...	5.8	2.48	...	5.8	2.01	...	4.8
798	1.64	...	6.0	1.44	...	3.5	1.23	...	3.6	1.30	...	4.3	1.34	...	3.3	1.69	...	4.1	2.50	...	5.4	2.99	...	7.3	2.56	...	6.3	2.70	...	5.7	2.64	...	5.0	1.70	...	6.2
799	1.26	...	2.9	1.09	...	3.3	1.14	...	2.7	1.18	...	3.1	1.69	...	4.7	2.00	...	4.2	2.67	...	7.9	2.83	...	5.7	2.02	...	4.9	2.07	...	4.4	1.69	...	4.4	1.61	...	3.3
800	0.99	...	2.7	0.83	...	1.9	0.95	...	2.4	0.85	...	2.2	1.49	...	3.6	2.10	...	4.9	2.46	...	5.7	3.06	...	8.3	2.00	...	4.8	1.55	...	3.9	1.09	...	4.0	1.31	...	2.7
801	1.54	...	4.8	1.10	...	4.3	1.03	...	2.6	1.01	...	2.6	1.21	...	3.0	1.44	...	3.7	1.81	...	5.7	2.12	...	5.7	2.85	...	7.0	2.22	...	7.4	1.95	...	4.7	1.39	...	3.5
802	0.52	...	1.0	0.40	...	1.1	0.30	...	0.7	0.40	...	1.0	0.78	...	3.1	1.42	...	3.8	2.61	...	5.9	2.34	...	6.1	1.52	...	4.4	0.80	...	3.1	0.74	...	2.2	0.50	...	1.1
803	0.29	...	1.0	0.25	...	0.9	0.21	...	0.4	0.29	...	0.8	0.54	...	0.7	0.74	...	1.6	1.22	...	2.0	1.07	...	2.4	0.94	...	1.6	0.67	...	1.2	0.44	...	0.6	0.28	...	0.6
804	2.23	...	5.0	2.37	...	6.6	2.30	...	5.2	1.83	...	6.5	1.88	...	3.7	1.80	...	4.4	2.57	...	7.7	2.81	...	6.2	3.33	...	10.4	3.04	...	6.8	2.85	...	6.5	2.87	...	6.6
805	3.35	...	12.4	2.72	...	7.7	2.29	...	5.5	2.00	...	5.1	2.06	...	5.5	2.07	...	6.2	2.55	...	5.1	3.05	...	9.8	4.44	...	12.8	4.02	...	9.6	4.09	...	11.2	3.08	...	7.6
806	1.25	...	3.0	1.09	...	3.4	1.23	...	3.6	1.35	...	4.1	1.52	...	3.0	1.91	...	4.4	2.67	...	6.8	3.21	...	7.1	3.90	...	8.1	2.41	...	6.2	1.72	...	5.0	1.56	...	4.1
807	1.54	...	4.9	1.34	...	3.7	1.42	...	3.7	1.50	...	4.0	1.57	...	4.8	2.01	...	5.9	2.40	...	5.7	2.64	...	6.7	2.00	...	4.2	2.24	...	6.7	1.97	...	5.9	1.77	...	6.0
808	4.26	1.6	10.8	3.16	1.2	5.8	3.14	1.1	6.0	2.29	1.1	5.0	2.09	1.3	5.4	1.78	1.4	4.9	2.29	2.6	5.8	3.01	2.8	5.8	3.01	1.7	8.1	3.95	1.7	10.1	4.25	1.8	7.8	4.79	1.8	7.6
809	6.72	...	10.9	5.28	...	9.7	4.88	...	10.1	3.66	...	6.4	3.35	...	6.5	2.60	...	6.2	3.15	...	8.6	3.62	...	8.1	4.72	...	10.6	6.14	...	12.0	6.48	...	11.4	6.61	...	13.7
810	4.95	...	14.2	3.94	...	9.2	3.20	...	11.9	3.18	...	8.8	2.78	...	9.4	2.57	...	7.2	2.61	...	7.1	3.16	...	9.5	4.88	...	12.4	4.96	...	15.1	4.32	...	10.9	5.87	...	13.0
811	5.76	...	10.8	4.84	...	9.8	4.44	...	9.4	3.84	...	7.8	3.17	...	9.8	3.28	...	5.8	3.11	...	5.9	3.06	...	8.9	5.70	...	10.5	5.80	...	13.9	5.22	...	9.6	5.45	...	9.8
812	2.80	...	7.2	2.58	...	8.6	1.96	...	6.5	1.50	...	4.9	1.39	...	5.2	1.53	...	4.1	1.48	...	3.6	1.62	...	5.1	2.72	...	6.5	3.03	...	6.5	2.54	...	5.0	2.53	...	6.1
813	0.62	...	2.3	0.66	...	4.4	0.65	...	2.9	0.49	...	1.5	0.57	...	3.9	0.70	...	2.3	1.12	...	5.1	1.28	...	6.8	1.34	...	3.1	1.36	...	7.8	1.08	...	6.1	0.73	...	2.6
814	3.46	...	11.0	1.73	...	5.2	2.20	...	5.5	2.40	...	7.0	2.75	...	9.2	2.09	...	4.2	2.13	...	6.7	2.48	...	6.1	4.05	...	3.9	6.20	...	10.6	3.42	...	11.1	2.08	...	9.1
815	0.42	...	2.1	0.51	...	3.6	0.67	...	3.8	0.59	...	3.7	0.55	...	2.4	0.51	...	3.1	0.91	...	5.3	1.10	...	4.1	1.06	...	2.7	1.10	...	3.5	1.06	...	4.1	0.51	...	2.7
816	0.35	...	1.1	0.28	...	1.1	0.47	...	1.8	0.47	...	2.4	0.55	...	2.7	0.83	...	3.5	1.22	...	3.2	1.42	...	4.2	1.33	...	5.9	0.94	...	3.5	0.67	...	2.2	0.31	...	1.7
817	1.46	...	6.2	1.81	...	9.9	1.77	...	6.0	1.10	...	4.8	1.65	...	6.5	1.30	...	5.4	2.22	...	8.1	3.11	...	11.0	3.27	...	7.6	2.48	...	9.2	1.89	...	5.5	1.87	...	8.0
818	3.27	...	14.0	2.72	...	13.0	3.39	...	15.1	2.44	...	9.2	3.58	...	11.6	3.03	...	9.7	3.27	...	8.5	3.78	...	10.3	6.02	...	13.0	5.87	...	14.5	4.41	...	20.6	3.07	...	12.9
819	4.70	...	9.8	3.92	...	8.1	3.35	...	7.7	2.96	...	6.2	3.33	...	6.8	2.44	...	6.5	2.11	...	7.5	3.18	...	9.4	3.00	...	8.6	4.42	...	9.4	4.78	...	7.8	4.81	...	9.9
820	2.82	...	7.1	3.06	...	7.4	2.30	...	5.7	2.00	...	6.9	2.20	...	4.4	1.33	...	3.6	0.80	...	2.8	1.47	...	9.5	3.42	...	7.3	3.28	...	6.7	3.50	...	6.5	3.10	...	7.0

APPENDIX II

REFERENCES IN TABLE 1 (APPENDIX I)

(1) U. S. Weather Bureau, Annual Summaries.

(2) U. S. Weather Bureau, *Bulletin W*.

(3) U. S. Weather Bureau, Climatological Data.

(4) World Weather Records, Smithsonian Institution, 1927.

(5) *Journal,* New England Waterworks Association, September, 1915; September, 1921; and June, 1926; "Rainfall in New England" by X. H. Goodnough, M. Am. Soc. C. E.

(6) *Monthly Weather Review,* April, 1926; "Climatological Data for the West Indian Islands", by W. W. Reed, U. S. Weather Bureau.

(7) *Monthly Weather Review,* Supplement No. 28, "Climatological Data for the Tropical Islands of the Pacific Ocean", by W. W. Reed, U. S. Weather Bureau.

(8) *Proceedings,* Institution of Civil Engineers (England), Vol. CCXVII, 1923-1924, p. 284, Hearn on "The Effect of Shape of Catchment on Flood Discharge".

(9) Etudes sur le Climat de la France (Extending to 1910), by M. Angot.

(10) "Precipitation in Northern Germany", Vol. 1, by Dr. G. Hellman, Berlin, 1906.

(11) "Precipitation in Italy", by Dr. Filippo Eredia, Stabilimente Giuseppe Civelli, Rome, 1908.

(12) *Geograyhical Review,* July, 1925, "Extreme Annual Range of Rainfall".

(13) "Relation of Rainfall to Configuration", by Carle Salter, Joint Director of the British Rainfall Organization, 1918.

(14) "Rainfall in Bulgaria", by Gencho Stainoff, Sofia, Bulgaria, 1924.

(15) Miscellaneous Compilations at U. S. Weather Bureau.

(16) "Climate of Palestine", by Felix M. Exner, Vienna, 1910.

(17) Climatological Studies of the Soviet Republic, Leningrad, 1927.

(18) Peterman's "Geographische Mitteilungen", Vol. XXVI, 1897, Paper No. 124.

"Rainfall Distribution Over the Earth's Surface", by Alex. George Supan.

(19) Climatological Normals for Egypt and the Sudan, Candia, Cyprus, and Abyssinia.

(20) "Rainfall in the Russian Empire", by H. Wild, St. Petersburg, 1887.

(21) *Memoirs,* Academy of Sciences, Vol. III, No. 1, by H. Wild, St. Petersburg, 1895.

(22) Black Sea Pilot, British Admiralty, Seventh Edition, London, 1920.

(23) "Die Vegetation der Schweiz", by Dr. H. Brockmann-Jerosch, Zurich, Switzerland, 1925.

(24) "Rainfall in the East-Pyrenees Department, 1851-1900", by Dr. Fines. Perpignan, France, 1902.

(25) Book of Normals of Meteorological Elements for the British Isles, Air Ministry, London, 1919 and 1923.

(26) "Rainfall of Northern Spain and Portugal", by Dr. Wilhelm Semmelbach, Hamburg, Germany, 1910.

(27) *Bulletin No. 102,* Netherlands Meteorological Institute, by Dr. Ch. M. A. Hartman, Utrecht, Holland, 1913.

(28) Philippine Weather Bureau *Bulletins.*

(29) Official Year Book of the Commonwealth of Australia, Melbourne, 1926.

(30) *Annales* du Bureau Centrale Meteorologique de France, 1893.

(31) "Rainfall Normals to End of 1925", by A. D. Lewis, Director of Irrigation, Union of South Africa, Capetown, 1927.

(32) Observations Pluviometriques, 1876, Vor Raulin, France, p. 177.

(33) Sitzungsberichte der Kaiserlichen Akademie der Wissenschaften, Vol. III, p. 75, 1902 (Long Period Rainfall Variation).

APPENDIX III

MAIN REFERENCES FOR TABLE 2

Atlas of American Agriculture, Pt. II, Section A, U. S. Weather Bureau, 1922, p. 48.

American Civil Engineer's Pocket Book, p. 1256.

"Water Supply Engineering", by A. P. Folwell, Third Edition, 1917, p. 131.

"Evaporation on United States Reclamation Projects", by I. E. Houk, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 90 (1927), p. 266.

"Principles of Irrigation", by F. E. Kanthack, p. 137.

Waterworks Handbook, by A. D. Flynn, R. S. Weston, and C. L. Bogert, Members, Am. Soc. C. E.

"Use of Water in Irrigation", by S. Fortier, M. Am. Soc. C. E.

"Design of Irrigation Structures", by E. A. Moritz, M. Am. Soc. C. E.

Publications of the Australian Commonwealth Bureau of Meteorology, 1914 and 1918.

DISCUSSION

GEORGE S. BLISS,[7] ESQ. (by letter).—The outstanding features of interest in this paper are the ten columns of percentage values in the tabulated matter and the discussion of precipitation patterns. The percentage columns shed new light on some old problems by placing them in new settings.

In the discussion of precipitation patterns a little more prominence might have been given to the part played by topography, which seems to be one of the major factors. Without more or less rugged topography the other factors lose much of their effectiveness. For example, with a topographic map of Pennsylvania, it is entirely feasible for one versed in the meteorology of this section to prepare a fairly accurate outline of the rainfall distribution and to estimate closely the relative differences between the wettest and dryest regions of the State without the aid of precipitation data. The author's deductions from generally accepted meteorological facts and theories seem good.

Steep barometric gradients are not always accompanied by steep temperature gradients. In the ordinary cyclonic disturbances it seems to require a combination of both conditions to produce maximum wind velocities. In instances where the temperature gradient is comparatively flat, the wind velocities are usually moderate despite fairly steep pressure gradients. The greatest wind velocities known are in tornado funnels, which are solely the result of great differences in temperature of adjacent air masses. The extremely steep barometric gradients in the funnels are results of, rather than causes of, the violent air circulations.

If there is any relation between variations in precipitation and sunspot numbers it is completely overshadowed in localities, or over small areas, by accidental causes. It could be detected only by computing mean monthly and annual rainfall over vast areas, such as a continent or an entire hemisphere. Variations in solar radiation, if there be such, are completely absorbed in the general circulation of the atmosphere, and are not in evidence in the temperature records of any particular region.

This paper as a whole seems to apply more particularly to the problems of flood forecasting, and less directly to those of flood control. Just to the extent that calculations for the latter are based on such stern realities as river-gauge heights and stream-discharge measurements, the variable factors leading up to that stage are eliminated, and accurately so.

BREHON B. SOMERVELL,[8] M. AM. SOC. C. E. (by letter).—The author has assembled a mass of basic data which should be of value for the general study of the factors responsible for rainfall and the corresponding run-off. Covering as they do the entire globe, these data should be helpful in analyzing any particular water-shed by comparison with other water-sheds.

Neither the data presented nor other generalities will suffice for a thorough study of a particular water-shed. If stream-flow records are lacking, considerable judgment must be used in drawing conclusions as to the resulting run-off because there are a number of factors affecting run-off, and also because there are different methods for combining these factors.

---

[7] Senior Meteorologist, U. S. Weather Bureau, Philadelphia, Pa.

[8] Maj., Corps of Engrs., U. S. A., Memphis, Tenn.

The factors themselves are fairly definitely known, and the effect of a definite factor, or a definite combination of factors, on the run-off to be expected from a definite amount of precipitation, can be predicted without too great an error. No method is known, however, whereby engineers can predict which combination of factors will occur at a specific time, and, consequently, estimates as to run-off will always be subject to considerable error for water-sheds of any appreciable size.

In studying an individual stream, the water-power resources, its navigable capacity, or the menace of its floods, the engineer is concerned particularly with the actual past performance of the stream. A mean of past performances is not sufficient, nor is the record from only one station of great value. Storage must be provided for low-water periods, during which the stream will not supply sufficient water for power or navigation, and storage or other means of flood control must be provided for the high-water period also when valley storage is not sufficient. For these purposes monthly quantities of flow are the smallest increments that may be used safely. If there is no record of these flows, so that they must be computed from rainfall data, then the records from all stations within the area under investigation should be included. For example, in the study of the Potomac River as a source of power the following observations are noted:

Mean Annual Precipitation, Potomac River Basin:	Inches.
From U. S. Engineer Office, approximately	41
From Table 1 (Appendix I), (Station 75, Column (13)), Washington, D. C	40.5
From Table 1 (Appendix I), (Station 76, Column (13)), Washington, D. C	42.2
Maximum Annual Precipitation Recorded in the Potomac River Basin:	
Bayard, W. Va., 1926	89.01
Maximum Annual Precipitation Recorded for Potomac River Basin in Table 1 (Appendix I):	
Washington, D. C. (Station 75, Column (14))	61.3
Washington, D. C. (Station 76, Column (14))	61.3
Elkins, W. Va. (Station 74, Column (14))	65.4
Minimum Annual Precipitation Recorded in Potomac River Basin:	
Western Port, Md., 1895	12.71
Minimum Annual Precipitation from Table 1 (Appendix I):	
Washington, D. C. (Station 75, Column (12))	18.8
Washington, D. C. (Station 72 (Column (12))	30.8
Elkins, W. Va. (Station 74, Column (12))	38.8

Maximum, minimum, and average tabulations at single stations are not conclusive as to conditions even in a relatively small surrounding area, and cannot be used with accuracy in the investigation of a near-by drainage area for water-power potentialities or flood-control studies.

As an instance the drainage basin of the Potomac River is covered in Table 1 (Appendix I) by the stations at Washington, D. C., and Elkins, W. Va. The maximum annual precipitation is recorded at Elkins, as 65.4 in. Actually, the maximum annual precipitation recorded within the Potomac River drainage basin was 89.01 in. at Bayard, W. Va., in 1926, a difference of approximately 24 in.

The same situation occurs in the minimum annual precipitation recorded as 18.8 in. at Washington, D. C., in Table 1 (Appendix I). Western Port, Md., however, in 1895, recorded an annual precipitation of 12.71 in., 6 in. less than the minimum recorded in Table 1 (Appendix I). An entirely different picture may thus be presented, depending upon the stations selected.

From these remarks it will be noted that although the mean for the water-shed derived from a large number of stations approximates fairly closely that given by Mr. Jarvis in his tables for the station at Washington, his maxima and minima are far from those which have been recorded at other stations within the past and also far from the maxima and minima applying to the basin as a whole.

In the study of large water-sheds involving several hundred thousand square miles, the data given in Table 1 (Appendix I) may furnish means, maxima, and minima closely approximating those which might be obtained from a much larger list of stations within the same area. In no case, however, is it believed that one station could be used with any degree of accuracy in a design of works for flood control, power, and navigation.

In other words, comprehensive summaries, such as the author has presented, aid one in studying the local problem by quickly giving one a broader geographical picture of the rainfall phenomena. They may keep one from drawing false general conclusions from limited local data and may lead the way to additional pertinent information. They do fall short, however, of giving those data essential to reliable conclusions as to the solution of a particular problem.

Jesse W. Shuman,[9] M. Am. Soc. C. E. (by letter).—This paper is a valuable contribution to the subject, with the points well brought out and the conclusions carefully worded. Undoubtedly, the views expressed are a résumé of the current opinions of both meteorologists and hydrologists. The attitude of the former is reflected in the statement, "no direct relation is traceable between rainfall and sunspot periodicity * * *"; and the attitude of the latter is indicated by the methods suggested for study of the rainfall data by means of frequency curves, the consideration of floods, etc.

Meteorologists are quite aware of the cycles in weather phenomena, but, in general, they make no application of this knowledge, probably because the day-to-day or month-to-month variation in any of the generally used meteorological data is negligible. Occasionally, papers appear in the *Monthly Weather Review* that deal with this subject authoritatively, from the viewpoint of meteorologists and mathematicians as well as "dyed in the wool" hydrologists. Stripped of all their involved mathematics, meteorological terms, and background, the crux of these papers is, that by proper treatment of sunspot numbers and rainfall or run-off data, a relationship does exist, which is easily discernible. The papers also show that, while these cyclic variations are negligible in their effect from day to day as to current weather changes, nevertheless, their effect is cumulative and works great changes in run-off, lake levels, etc.

---

[9] Secy-Treas., Power Eng. Co., Minneapolis, Minn.

The writer does not hereby intend to decry the work of the author; rather he wishes to show in what way the valuable data assembled by Mr. Jarvis can be further considered and used to great profit.

The author's Fig. 2 compares a graph of sunspot numbers with graphs of several rainfall stations. From an inspection of the graphs, Mr. Jarvis draws the conclusion that there is no traceable relation between the sunspots and the rainfall. The method of comparison here used is perfectly standard in its application, and it is readily admitted that, with the graphs as drawn and located, there is no easily recognizable relation.

In Fig. 8, the same data for sunspot numbers are used,[10] and graphs of rainfall at Milan, Italy, New Orleans, La., Charleston, S. C., and Greenwich, England, are shown. In Fig. 2 the sunspot numbers are inverted, with zero numbers at the top; whereas Fig. 8 shows the zero numbers at the bottom. Furthermore, Fig. 8 has a much larger vertical scale. After plotting all the sunspot numbers (the mean Wolf and Wolfer provisional number for the year) for the entire record, a smooth line is drawn through the plotted points. This yields the series of loops shown. This curve, thus far, is identical with Fig. 2, except that it is inverted. Each peak or crest represents maximum sunspot numbers, and the troughs represent minimum sunspot numbers. When viewed in this state, there is no apparent relation with rainfall. However, the curve may be further reduced. Divide the distance between each peak and trough (as at 1 and 2) with a dot; then pass a smooth line, Curve *B*, through the dots. This curve is the well-known Bruckner cycle; it is merely the median line traced through the smoothed sunspot numbers, which is the same line one would obtain if the sunspot numbers graph were repeatedly integrated. The peaks of this cycle represent, of course, relatively higher sunspot numbers. They occurred in 1787, 1803, 1837, 1870, 1893, and 1917.

Bruckner's discovery of this cycle was accomplished from studies[11] (mostly of European data) of secular variation of the Caspian Sea; lakes and seas without outlet; river heights; precipitation; atmospheric pressure; temperature; and times of grape harvest.

Although suspecting a relation between solar variation and terrestrial phenomena, Bruckner's efforts in this direction were unsuccessful. He simply reduced his statistical data by the use of five-year running means, and this method is insufficient to segregate the cycles.

Meteorologists discarded the cycle because it gave no reliable clue as to present-day variation in weather; and, further, because there was apparently no connection between it and any great "cause". With the advent, however, of the publications of the U. S. Weather Bureau, methods of analysis, with examples and illustrations, were given, showing more clearly the nature of the Bruckner cycle.

---

[10] From *World Weather Review* and *Monthly Weather Review*.

[11] "Klimaschwankungen seit 1700," 1890, Vienna.

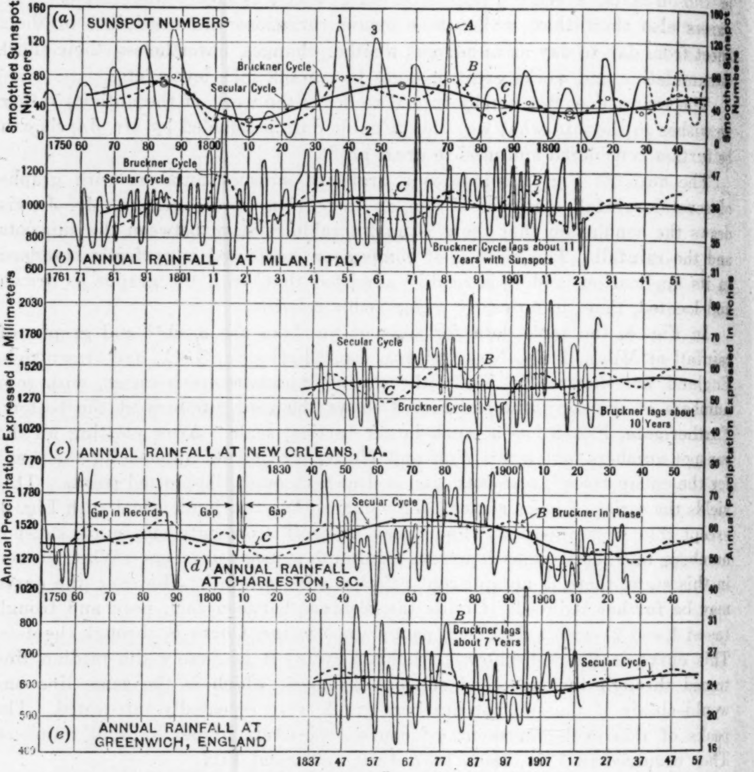


FIG. 8.—COMPARISON OF BRUCKNER CYCLES FOR SUNSPOT NUMBERS WITH CYCLES OF MAXIMUM RAINFALL

This cycle is now known to have a periodicity of about 22.6 years (crest to crest), superimposed upon a longer cycle called the Secular cycle. According to "Reports of the Conference on Cycles",[12] Hale's double sunspot period of about 22.5 years has been faithfully recurring (with slight variation) since 1810.6 for sunspot minima, and since 1816.4 for sunspot maxima. These cycles provide a very fair basis for predicting that 1946 will be a year of minimum sunspots; and also that 1951 will be a year of maximum sunspots. Using the same period of 22.6 years, and working from the sunspot minimum of 1913.6, the occurrence of the next probable sunspot minimum is predicted in 1936. These future periods have been shown extended on Fig. 8 (*a*).

---

[12] Carnegie Institution of Washington, 1929.

The Secular cycle is the next longer swing cycle mentioned by Bruckner (see Curve *C*, Fig. 8). This cycle is also variable in length, the last three periods being estimated at 70, 60, and 90 years, or an average of about 73 years. The cycle was high in 1855 and low about 1906. It is now trending upward toward a peak which may occur about 1945 to 1950.

Thus, in the provisional sunspot numbers, the value changes enormously from month to month, although the mean annual value has a periodicity of about 22.6 years from one maximum to the second next maximum. There are several cycles in these sunspot numbers, but the most important, for the present purpose, are the Bruckner and the Secular cycles. The result is to provide a cycle or pattern curve, constructed from sunspot numbers, and the next task is to show what use can be made with it.

Refer now to the rainfall graphs in Fig. 8. Of course, it is recognized that a single gauge does not truly register the rainfall at any district; that, in order to have an accurate measure, it would be necessary to have several gauges distributed over the district. When the data from several stations are averaged, the fortuitous storms are more or less "ironed out". Where only one station is considered, as in Figs. 2 and 8, the cycles may not emerge as clearly as they would if the graph were made up of data averaged over an area of several hundred square miles. On the other hand, if too large an area is taken, the data at one end or part of the district may neutralize those of another section, and one may secure no recognizable cycles. Preferably, then, eight or ten Weather Bureau rainfall stations should be included, and their data should be averaged. After one has gained experience in this type of analysis, it is possible to trace in the cycles in the data of a single rainfall station, as is done in Fig. 8.

Consider the graph of Milan rainfall (Fig. 8(*b*)). Here is a long record that, by proper treatment of its data, can be made to show both the Bruckner and Secular cycles. The treatment consists, essentially, of smoothing processes, and separation of the original graph into its component parts.

Assume, therefore, that these processes have been gone through with the Milan rainfall data and that, finally, the Bruckner cycle has been derived. Plot this cycle in the original graph, and then compare the graph with that of the sunspot numbers. In order to get the best synchronism between the Bruckner cycles in the two graphs, it will be necessary to set 1881 on the Milan graph on the 1870 ordinate of the sunspot graph. No attention is given to the amplitudes of the cycles; only the synchronism of the peaks and depressions is important. Fig. 8 shows that the trend in rainfall at Milan lags behind that of the sunspot numbers by about eleven years. If one looks at the Milan graph trying not to see any cycles, it seems only like a series of oscillating values, with no law or order. With the cycles drawn in, however, it certainly is obvious that, in general, the trend of wet and dry periods is in accordance with the "ups" and "downs" of the cycles.

In attempting to compare rainfall or run-off graphs with the comparison curve, that is, with the sunspot Bruckner cycle, the data of one cannot be compared with the current data of the other; there may be a lag or a lead; only occasionally are both sets of data in phase with each other. Bruckner thought that the phase of his cycle was the same all over the world, but now it is known that the phase varies, depending on the geographic locality; that is, along the Atlantic Coast, at Boston, Charleston, etc., the trend of rainfall is about in phase with the sunspot numbers. Toward the West, however, the phase begins to lag, until a maximum lag occurs at the Pacific Coast. The phase may be different even within the confines of a single State. The prime requisite, therefore, in making comparisons of rainfall graphs is to determine first the phase of their trends.

The Secular cycle in the Milan graph is high in 1855 and low in 1906, the same as it is in the sunspot graph, because the phase of this cycle seems to be the same all over the world. Fig. 8(*b*) shows where the highest values for total annual rainfall occurred; that is, from 1841 to 1851, which is at the crest of the Bruckner, and near the crest of the Secular cycles. One expects higher values at or near the crests of these cycles, and, of course, lower values at the troughs. The graph indicates that the Bruckner cycle is the major control (because such low values occurred due to the Bruckner trough in 1861 to 1871, even if this period was very near to crest of the Secular cycle). A Bruckner crest is indicated in 1930, so that moisture conditions should now be above normal; and the next great wet period is indicated about 1945 to 1955.

In Fig. 8(*c*) the Secular cycle of rainfall in New Orleans is quite flat, with small amplitude. The Bruckner cycle is responsible for the great swings in moisture conditions in this case. Note that 1880 of the New Orleans scale is set on the 1870-year ordinate of the sunspot graph, thus indicating a lag of about ten years at that point.

In Fig. 8(*d*) the Bruckner cycle for rainfall at Charleston, S. C., is in phase with the sunspots trend, as indicated. The maximum wet period of the record occurred after the crest of the Secular cycle. The graph shows a trough of the Bruckner cycle at 1929, indicating a control toward lower rainfall values. Since there has been such a great drop from 1924 to 1927, the rainfall for 1928 is more than likely to be quite high. A study of rainfall records shows that Nature tends to preserve its balance, as it were; and that extreme values are likely to be followed by extreme values of opposite sign the following year or period.

Finally, Fig. 8(*e*) is for the rainfall at Greenwich, England; the lag is about seven years. The Bruckner cycle was at crest in 1927, indicating moisture conditions above normal. These few examples should suffice to show that:

(1) Rainfall has a distinct, traceable relationship with sunspots.
(2) Proper attention must be given to choice of vertical and horizontal scales used for plotting; otherwise, the cycles, whose amplitudes of swing may be very slight, will not easily be discernible.
(3) All rainfall graphs follow the same general pattern, even if they may differ materially in the limits of their departure from the mean value.
(4) After the phase of the trends of rainfall has been determined, with respect to the long swing curve in the sunspot numbers, a great similarity between the different rainfall graphs is observed. Rain-

fall all over the world is brought about by the same "causes", even though the lapse of time before the "effect" takes place, varies.

While the foregoing remarks have been applied to rainfall they may cover run-off data as well. In fact, run-off data give much better cycles, as the drainage areas act as integrators. The advantage of being able to identify these cycles is simply that they provide an index of the trends of the graphs, which is of value for bridging gaps in the past, and for indicating the probable times of future peaks or troughs, some years in advance. The extreme of rainfall, as listed in Table 1 (Appendix I), can now be used to greater advantage, especially if the year and month of their occurrence is noted.

If a hydrologist knows when to look for a peak in the Bruckner cycle in rainfall data at a certain locality, he has a good idea of the period of flood occurrence, referring to the great floods. Floods do occur at or near the troughs of the Bruckner curve, but they do not reach the magnitude of the others. Each section of the country needs to be studied individually. The times of flood occurrence can be marked on the Bruckner cycle in the rainfall graph, and note can be made of the position of such occurrence. It is believed that greater accuracy as to flood occurrence and magnitude can be secured by this type of study than by treating all flood data by means of frequency curves and probability methods.

A. Streiff, M. Am. Soc. C. E., has shown how to estimate the total run-off on a river for future years if twelve years of monthly means, determined by fairly good gauging, are available.[13] Considering also that most rivers lag behind the rainfall over their drainage areas from one to five months, it is quite obvious that hydrologists need no longer remain in the dark as to the total annual water output of the river for the next year. The extremes of discharge during the year, constituting floods, require much further study.

The fact of lag or lead in phase of the trends of rainfall, with the common great cause, explains why local floods occurred in Alabama in the autumn of 1929, while at the same time the region around Seattle, Wash., was suffering from one of its worst droughts. Each section of the country, part of a State, etc., is performing its cycle of operations, giving now wetter and now dryer weather, in accordance with these cycles. If the troughs or crests of the Bruckner and the Secular cycles happen to come close to phase, the extremes of rainfall occur. If the parts of the country which feed rivers that eventually come together (such as the drainage areas of the Mississippi River) happen to have cycle phases that practically coincide, then the great floods such as that in 1927, occur.

The graph of Greenwich, indicates a peak of the Bruckner cycle about 1929 (Fig. 8(*e*)). On January 7, 1928, one of London's worst storms occurred. The graph of New Orleans, La. (Fig. 8(*c*)), indicates a Bruckner cycle crest about 1926. On September 8, 1929, this city had a great storm, with 10.75 in. of rain falling in 24 hours.

It is conceivable, then, that the great floods of the Mississippi are the results of the phase coincidence of these important cycles. The writer suggests that the Mississippi River Commission make an analysis along the lines outlined, studying each tributary river first, then combining the results for a grand total and a grand resulting series of cycles. Noting on these curves the flood times that have occurred in the past, the Commission should have much more data by which to predict future floods than it has by means of previous studies that have all been based on the theory of probability. Probability deals with numbers that occur purely by chance. It is the writer's opinion that rainfall and run-off data are not chance numbers, but are related to a great "cause".

---

[13] *Monthly Weather Review*, March, 1928.

CLINTON L. BOGERT,[14] M. AM. SOC. C. E. (by letter).—Only those who have put in tedious hours, days, and weeks in compiling, classifying, weighting, and averaging long-term rainfall and stream-flow records can really appreciate the immense amount of drudgery which the author has undergone in producing Table 1 (Appendix I). The profession should be deeply obligated to him for the form and extent of the results.

In this table the author has attained his desideratum of rigorous condensation, at a sacrifice of some essential data. The usability of the data in localities where stream-flow records are published, would have been increased materially were the year entered in the record when each minimum and maximum rainfall (both yearly and monthly) occurred. The dates would also be of value in investigations of droughts and amplitude of rainfall cycles.

In using any observed data, such as rainfall, there is one element of error which is often overlooked: All observers are assumed to make and record their observations with equal conscientiousness and accuracy. Of course, this is humanly impossible. The observations from which Table 1 (Appendix I) is made up, run into the millions, and to all, the same accuracy is assigned. The author's method of expressing results in percentages will reduce this error of observation and recording to some extent.

The value of Table 1 (Appendix I) for further research would have been enhanced had there been space available for assigning to each of the 820 stations a specific reference as to the source of the data.

R. W. POWELL,[15] ASSOC. M. AM. SOC. C. E. (by letter).—In making this complete compilation of rainfall data the author has rendered a great service to the Civil Engineering Profession, and his discussion of the data is also helpful. The writer wishes merely to comment on two or three of the many phases treated.

The author states that "no direct relation is treaceable between rainfall and sunspot periodicity", which seems to dismiss too lightly the results of the extensive study that has been put on this subject. The writer would not go as far as such extreme "cyclists" as Alter,[16] Streiff,[17] or Gillette,[18] but it seems that there must be some truth behind their computations. The studies of men like Huntington[19] and Clayton[20] seem to establish a scientific basis for a relationship between solar radiation and rainfall.

---

[14] Cons. Engr. (Sanborn & Bogert), New York, N. Y.

[15] Asst. Prof. of Mechanics, Ohio State Univ., Columbus, Ohio.

[16] "An Examination by Means of Schuster's Periodograms of Rainfall Data from Long Records in Typical Sections of the World", by Dinsmore Alter, *Monthly Weather Review,* February, 1926, pp. 44-56; also, several other articles in the same periodical.

[17] "Sunspots and Rainfall," by Abraham Streiff, M. Am. Soc. C. E., *Monthly Weather Review,* February, 1927, pp. 69-71, and a former article in the same periodical for July, 1926, pp. 289-296.

[18] "Five Important Weather and Earthquake Cycles," by H. P. Gillette, M. Am. Soc. C. E., *Water Works and Sewerage,* October, 1929, pp. 429-432, and "A 22.4-Year Cycle in Rainfall and Its Cause", by the same author, in the same periodical for November, 1929, pp. 477-480.

Some studies on this subject have convinced the writer that the variations in annual rainfall do not take place purely at random but must be regarded as random variations of fairly large amplitude superimposed upon several cyclic variations of smaller amplitude. For example, the following equation was deduced for Boston, Mass.:[21]

$$y = 44.22 + 8.60 \cos \frac{k\,(x - 1767)}{96} + 2.58 \cos \frac{k\,(x - 1773)}{47}$$

$$+ 2.70 \cos \frac{k\,(x - 1798)}{34} + 2.56 \cos \frac{k\,(x - 1811)}{18} + 1.88 \cos \frac{k\,(x - 1810)}{10} \ldots (1)$$

in which, y is inches of rainfall to be expected in any given year; k is 360 to give the angle, in degrees, or 2π to give it, in radians; and x is the date (A. D.) of the year considered.

The 111 years of record from 1818 to 1928, inclusive, fitted this equation with an average deviation of 4.28 in., while the average deviation from the mean was 6.36 in. Equation (1) is not recommended for use in forecasting the rainfall at Boston; it is given simply to illustrate the fact that after the best combination of cycles has been selected to fit any given set of data, the individual deviations will still be large. However, the cycles concealed by these random deviations may still be real, and the writer believes that one of them will usually have a period of about eleven years, and will synchronize with the variations in sunspots. Equation (1) is purely empirical; the denominator of the last fraction could have been changed to 11 or 11.2; some of the coefficients could have been changed slightly, and the equation would have fitted the data nearly as well.

The other point upon which the writer wishes to comment is variability or dispersion of the amounts of annual rainfall at a given station, when no account is taken of the order of occurrence. When the figures for annual rainfall are arranged in the order of magnitude, what is the result? The author has touched upon this question by presenting Fig. 3, which is a semi-logarithmic plotting of part of the data for four stations; but for the remainder he has indicated the variability only by giving the maximum and minimum of the observed values. This is manifestly a very uncertain method of describing the variability, because it depends on only the two measurements and neglects all the others; and also because it depends quite a little on the length of the period of record. As Fig. 3 shows, if 20 years of record have indicated a certain range between maximum and minimum, it is probable that 200 years of record at the same station will show a range of about twice as much. If the data for all the stations were plotted in this same way the lines for the stations where the annual rainfall is most variable would be steepest and the slope, or some number depending upon it, could be taken as a measure of the variability.

---

[19] "Earth and Sun," by Ellsworth Huntington, and "Climatic Changes," by Huntington and Visher.

[20] "World Weather," by H. H. Clayton, MacMillan, 1923, especially the section on "Relation of the Sunspot Period to Rainfall", pp. 309-314; the map on p. 264; and the chapter on "The Sun and the Weather", pp. 215-269.

[21] See an article by the writer in *Annals of Mathematical Statistics*, May, 1930.

In a paper entitled, "Flow in California Streams"[22] data are given for annual rainfall for about 260 California stations for the 50 water years, 1871-72 to 1920-21 (or for those of them that are known). Rainfall is expressed both in inches and in percentage of the average rainfall for those years. For a supplementary list of 17 stations the data are given in inches only; 41 of the stations in the first group are included in the author's list. To these, the writer added 4 from the supplementary table and for each of these 45, he plotted a graph. Figs. 9 and 10 are the plottings for Stockton, Folsom, and Sterling. These show the method and illustrate one of the less variable stations, one more nearly the average, and the most variable, respectively. The method of plotting is slightly different from that adopted by the author. The curves are turned so that the numbers increase from right to left instead of left to right and the serial number of the order of magnitude is plotted instead of "average period between occurrences of a given amount." This gives the same curve as the author's method without the trouble of computing the average periods or of plotting fractional amounts.

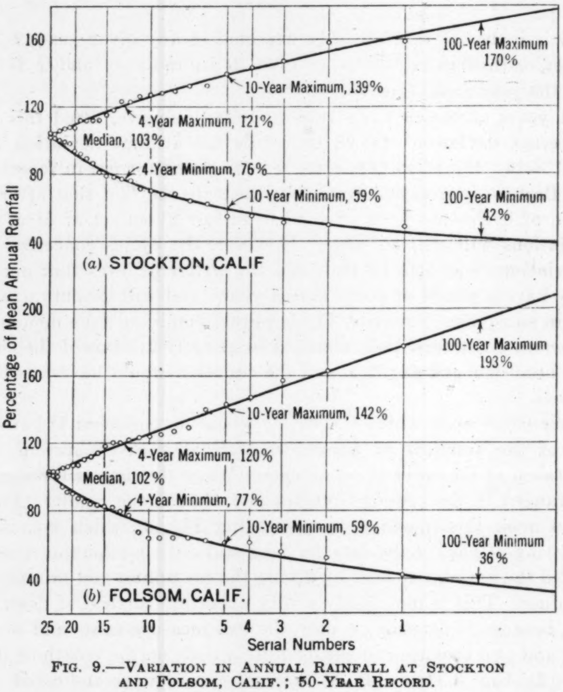


FIG. 9.—VARIATION IN ANNUAL RAINFALL AT STOCKTON AND FOLSOM, CALIF.; 50-YEAR RECORD.

---

[22] *Bulletin No. 5,* Dept. of Public Works, Div. of Eng. and Irrig., State of California.

From these curves the expected maxima and minima every 4, 10, and 100 years, and the median values, were read and tabulated in Table 6. The 4-year maxima and minima are the quartile points, such that one-half the values come between them, one-quarter above, and one-quarter below. If the variation were according to the "normal law of error", half the difference between the 4-year maxima and minima would be the "probable error" or the standard deviation multiplied by 0.6745. At first thought, this would seem the best constant by which to measure variability; but it was found that stations with about the same range between 4-year maxima and minima displayed very different ranges between 100-year maxima and minima. However, as the period of record averages only about 40 years, the 100-year values are somewhat uncertain. It was thought best, therefore, to note the 10-year maxima and minima and to take their difference as a measure of the variability. This difference is tabulated in Column (11), Table 6, and the stations are arranged in the order of increasing variability as thus measured. Column (1) gives the number of the station corresponding to Column (1) of Table 1 (Appendix I). (It should be noted that the number of years of record is almost always less than that given

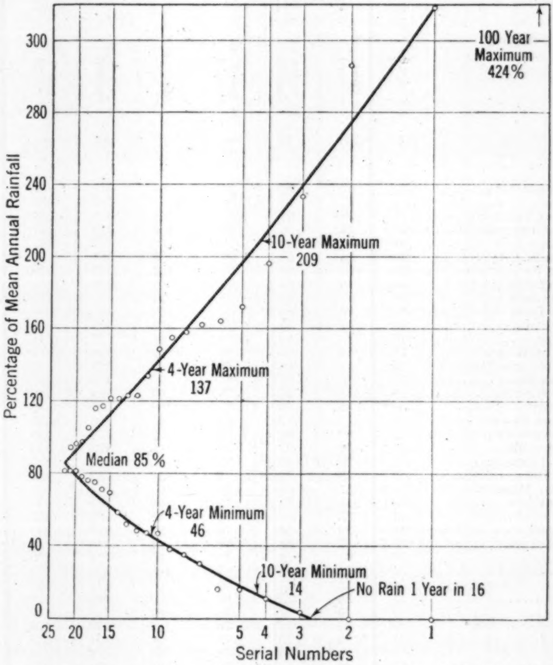


FIG. 10.—VARIATION IN ANNUAL RAINFALL AT STERLING, CALIF.; 45-YEAR RECORD.

by the author, because the writer has generally included only data between the years 1871 and 1921). The values given in Table 6 are sufficient to enable one to reproduce the curve, so that the expected variation in any other period of years desired may be determined with an accuracy about as great as is justified by the original data.

TABLE 6.—VARIATION IN ANNUAL RAINFALL AT FORTY-FIVE CALIFORNIA STATIONS AS READ FROM SEMI-LOGARITHMIC PLOTTINGS OF THE DATA.

Station No.	Station.	Years of record.	Minimum Precipitation.			Median value.	Maximum Precipitation.			Variation, for 10 years of record.
			100 years of record.	10 years of record.	4 years of record.		4 years of record.	10 years of record.	100 years of record.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
214	Crescent City	30	62	72	81	94	117	139	183	67
216	Eureka	34	51	70	80	92	115	138	184	68
225	Mount Tamalpais	22	28	64	82	101	118	134	175	70
253	Lick Observatory	40	44	66	80	100	118	137	178	71
260	Auburn	50	44	64	79	101	122	137	156	73
261	Nevada City	50	53	63	77	100	121	136	157	73
262	Lake Spaulding	27	50	63	77	101	123	138	162	75
257	Placerville	50	43	63	77	98	120	138	167	75
256	Tamarack	18	57	65	75	91	113	141	211	76
223	Fort Ross	45	39	64	80	102	123	141	177	77
254	Stockton	50	42	59	76	103	121	139	170	80
250	Fresno	40	46	68	79	96	122	148	200	80
234	Mount Wilson	17	42	67	82	99	129	147	166	80
255	Mokelumne Hill	36	57	64	76	102	125	146	198	82
222	Santa Rosa	33	36	65	81	99	120	147	215	82
215	Fort Gaston	20	56	66	78	99	124	149	200	83
259	Folsom	50	36	59	77	102	120	142	193	83
249	Lemon Cove	21	57	72	83	95	123	157	243	85
217	Weaverville	31	50	58	74	92	119	145	194	87
220	Marysville	50	39	57	73	96	123	146	192	89
226	San Francisco	73	29	60	78	96	119	149	225	89
241	San Bernardino	50	45	53	71	98	122	143	190	90
221	Helen Mine	21	51	60	75	105	137	151	166	91
238	Cuyamaco	33	43	61	75	96	131	152	172	91
219	Red Bluff	44	37	62	78	98	127	153	205	91
252	Merced	49	30	58	76	97	121	149	203	91
229	Monterey	37	37	58	72	94	130	149	212	91
264	Truckee	50	31	58	75	96	124	150	202	92
246	Bakersfield	31	31	63	84	105	130	156	212	93
233	Lowe Observatory	21	20	47	66	95	124	141	161	94
237	Campo	31	36	55	73	99	128	151	163	96
218	Delta	39	40	54	70	101	125	150	214	96
228	Boulder Creek	28	22	57	78	102	126	154	215	97
227	San Jose	47	23	55	74	96	124	153	217	98
258	Sacramento	73	29	52	72	95	121	152	220	100
230	San Luis Obispo	50	29	54	79	92	126	155	205	101
236	San Diego	72	35	51	68	95	123	153	217	102
245	Tehachapi	37	36	53	70	98	129	161	228	108
232	Los Angeles	44	32	51	68	96	127	160	245	109
231	Santa Barbara	50	30	51	67	90	129	162	209	111
235	Riverside	40	37	49	65	96	180	162	228	113
248	Independence	29	28	51	69	101	147	203	342	152
240	Indio	43	10	28	48	90	150	190	217	162
244	Mohave	37	*	19	48	100	146	193	311	174
239	Sterling	43	†	14	46	85	137	209	424	195

* No rain 1 year in 31 years (on the average).

† No rain 1 year in 16 years (on the average).

The curves given in Figs. 9 and 10 are typical and show that the line resulting from this semi-logarithmic plotting is not generally straight. Some investigation was made of other methods of plotting. Hazen's arithmetic probability paper,[23] gives a line more curved than the semi-logarithmic. Hazen's logarithmic probability paper[24] is still worse. Goodrich's skew probability paper[25] was also tried and seemed about equal to Hazen's arithmetic paper. Hall's hydraulic probability paper[26] gave an almost straight line, as was to be expected since it was derived from the flow of California streams; but when data for the Northeastern United States and Northwestern Europe were plotted, it did not give as straight a line.

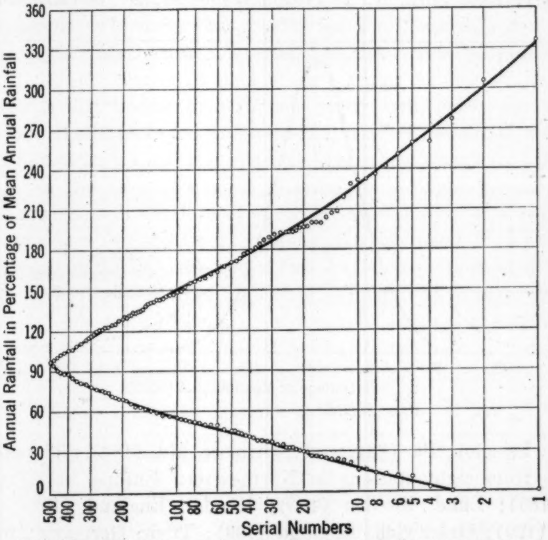


FIG. 11.—VARIATION IN ANNUAL RAINFALL AS ILLUSTRATED BY COMPOSITE DATA FROM TWENTY CALIFORNIA STATIONS, TOTALING 1 000 YEARS OF RECORD.

A further study of the slope of the frequency curves was made by using the composite data from twenty of the stations, which gave a total of 1 000 years of record. The stations used, in ascending order of variability, were Auburn, Nevada City, Placerville, Stockton, Folsom, Marysville, San Francisco, San Bernardino, Merced, Truckee, Bakersfield, Sacramento, San Luis Obispo, San Diego, Los Angeles, Santa Barbara, Independence, Indio, Mohave, and Sterling. It must be admitted that this selection is somewhat overweighted with stations of high variability and that the extreme values from a few stations appear very prominently, so that the resulting curve (Fig. 11), obtained by using these 1 000-year data as though they were all obtained at one station, is hardly typical of the State as a whole, but of its more variable portion. In other words, the middle portion of the curve, excluding, say, the upper 50 and lower 50 of the 1 000 values, is fairly typical of the State as a whole, while the "tails" are typical of the most variable stations.

---

[23] *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 220.

[24] *Loc. cit.*, p. 221.

[25] "Straight Line Plotting of Skew Frequency Data," by R. D. Goodrich, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 91 (1927), pp. 1–118.

[26] "The Probable Variations in Yearly Run-Off as Determined from a Study of California Streams", by L. Standish Hall, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 191-257; Fig. 4, p. 211.

In Fig. 12, the frequency distribution of the California record is compared with that of two other series of composite rainfall records totaling 1 000 years of record each.[27] One consists of records from ten stations in the Northeastern United States, as follows (years of record given in parentheses); Boston (110); Lowell, Mass., (99); New Bedford, Mass. (108); Providence, R. I. (93); Albany, N. Y. (103); New York, N. Y. (103); Rochester, N. Y. (93); Philadelphia, Pa. (103); Lebanon, Pa. (82); and Baltimore, Md. (106). The other consists of records from eight stations in Northeastern Europe, as follows: Paris, France (186); Lund, Sweden (160); Kendal, England (135); Edinburgh, Scotland (120); Greenwich, England (109); Tilsit, Germany (105); Copenhagen, Denmark (100); and Brussels, Belgium (85). The points for Northeastern United States and Northwestern Europe agreed so nearly that one curve was drawn to represent them both.

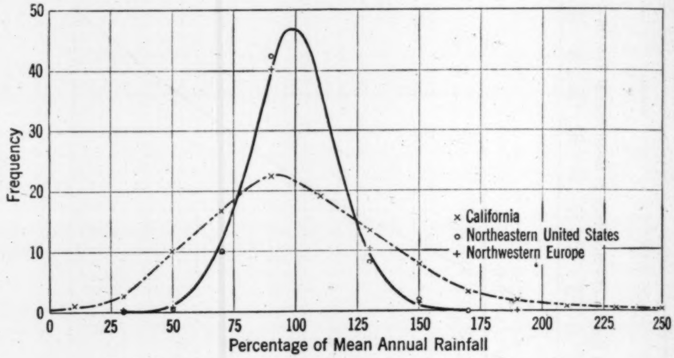


FIG. 12.—COMPARISON OF RAINFALL FREQUENCY RECORDS.

TABLE 7.—COMPARISON OF RAINFALL FREQUENCY RECORDS.

Symbol.	Definition.	California stations.	Northeastern United States and Northwestern Europe.
σ	Standard deviation, in percentage of annual rainfall.	40.3	17.1
$\sqrt{\beta_1}$	Asymmetry, or skewness	0.940	0.226
β_2	Flatness..	5.69	3.64
J	Type criterion..................................	− 2.73	− 1.13

These data were also investigated by the methods given by Fry.[28] Using his notation, the data gave the values listed in Table 7.

---

[27] From data in *Monthly Weather Review*, 1924, p. 485

[28] "Probability and Its Engineering Uses," Chapters VIII and IX.

These distributions (Table 7), therefore, can be represented by Pearson's Type IV, or by a Gram-Charlier series, or perhaps by some form not considered by Fry, such as that of Krichevsky.[29]

While no great importance can be attached to the values for California because the data are not really homogeneous enough to be combined, it is interesting to note that the California streams investigated by Hall[30] gave a "coefficient of variation" of 0.48, while the rainfall gives 0.403 (since the mean is 100% and "coefficient of variation" is defined as standard deviation ÷ mean).

H. R. Tolley states[31] that several hundred years of data are necessary before the fourth moment (which is what determines the "flatness") can be determined accurately. This is probably true, but the writer believes that it is allowable to build up composite data from stations having similar characteristics. The writer thinks that further study will show that ordinarily rainfall and run-off will be found to have a "flatness" of more than 3 (the value for "normal" distribution) and that extreme variations, therefore, will be more frequent than in the case of normal distribution. While normal distribution is the only kind with which most engineers are familiar, it is really a very special case that does not occur very commonly in Nature. If it proves to be true that extreme variations are more frequent than a normal distribution would call for, this fact may have important applications in investigating the probabilities of floods and droughts. For example, perhaps the 1894 flood on the Columbia River[32] was not so unusual as was thought.

The author's Fig. 4 is extremely interesting and one cannot but conjecture whether if, instead of "maximum rainfall observed" for the various periods at the various stations, one could plot "maximum to be expected" in, say, 10 years, one would not get even more consistent results.

Glen N. Cox,[33] Jun. Am. Soc. C. E. (by letter).—The results of the author's hydrologic studies are very valuable, especially since the rainfall records from all parts of the world have been studied and analyzed.

There is one consideration that should be kept in mind. With such an exhaustive study as this, the final results may not represent the facts in every detail. For example, in Fig. 13, the maximum rainfall of Iowa is shown as varying from more than 4 in. in the southeastern part to more than 7 in. in the northeastern and southwestern parts.

The values in Fig. 13 were taken from the Weather Bureau records published by the U. S. Department of Agriculture. These results show that an erroneous impression might be obtained from Fig. 5 as presented by the author. Bonaparte, Iowa, is in the southeastern part of the State, practically on the 5-in. isohyetal line in Fig. 5; yet the records show a measured rainfall of 12.10 in. from this station. Larrabee, with its 12.99 in. of measured rainfall, would be practically on a 6-in. isohyetal line. In Fig. 13 note the widespread distribution of the stations which have had measured rainfalls in excess of 9 in.

---

[29] "On Krichevsky's Method of Fitting Frequency Curves," by Edgar W. Woolard, *Monthly Weather Review*, February, 1924, pp. 91–94.

[30] *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 191.

[31] "Frequency Curves of Climatic Phenomena," *Monthly Weather Review*, 1916, pp. 634–642.

[32] *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 223.

[33] Assoc. Prof., Dept. of Civ. Eng., Louisiana State Univ., Baton Rouge, La.

It is quite evident that such a study as that presented by the author can only furnish general indications and that the records must be consulted for detailed information on the locality in question. It would sometimes prove disastrous if engineering works were designed to handle such low values of rainfall as those shown in Fig. 5 for the State of Iowa.

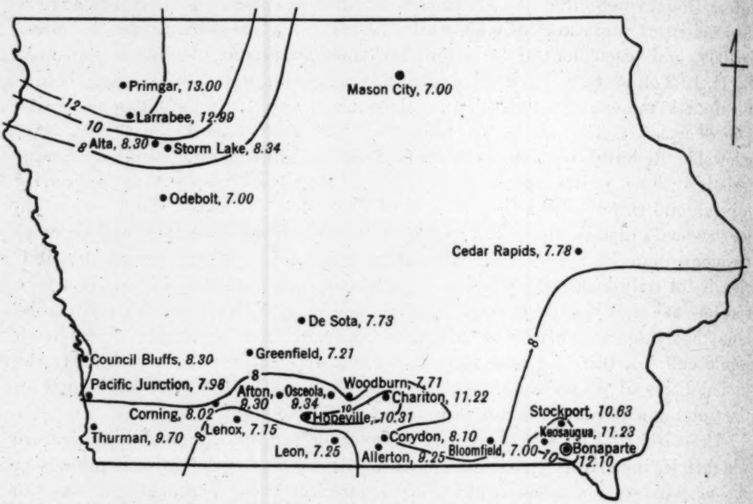

Primgar, 13.00
Larrabee, 12.99
Alta, 8.30
Storm Lake, 8.34
Odebolt, 7.00
Mason City, 7.00
Cedar Rapids, 7.78
De Sota, 7.73
Council Bluffs, 8.30
Greenfield, 7.21
Pacific Junction, 7.98
Afton, 9.30
Osceola, 9.34
Woodburn, 7.71
Chariton, 11.22
Corning, 8.02
Hopeville, 10.31
Thurman, 9.70
Lenox, 7.15
Leon, 7.25
Corydon, 8.10
Allerton, 9.25
Bloomfield, 7.00
Stockport, 10.63
Keosauqua, 11.23
Bonaparte 12.10
12
10
8
8
8
10
8
10

FIG. 13.—MAXIMUM 24-HOUR RAINFALL IN THE STATE OF IOWA, IN INCHES.

C. R. PETTIS,[34] M. AM. SOC. C. E. (by letter).—Table 1 (Appendix I) is a distinct contribution to the subject of meteorology, and should be in the library of every engineer who is interested in the subject of river flood flow. It contains a great deal of information in convenient form, and from it an engineer can obtain some idea about the rainfall characteristics of almost every place in the world. The author has performed a difficult task, and he is to be congratulated on the result.

The comments made in this discussion are based on the relation of rainfall to river flood flow, and with particular reference to maximum precipitation and maximum floods. In most flood-control work hydrologists are mainly interested in the maximum quantity of rain to be expected in one storm. The book entitled, "Storm Rainfall of Eastern United States",[35] is an excellent study of storms in relation to their possible flood-producing qualities. In this report storm precipitation is considered for various periods of time, from one day to six days. In Table 1, the author gives precipitation data for one hour, one day, and one month; of these three periods the one-day precipitation gives the best indication of what may be expected in one storm.

---

[34] Lt. Col., Corps of Engrs., U. S. A., Fort Logan, Colo.

[35] Technical Rept., Pt. V, Miami Conservancy Dist.

In Table 1 (Appendix I), the flood-control engineer may use the 24-hour precipitation as an index, but he should keep in mind that the actual rainfall in one storm will generally be greater than the precipitation in 24 hours. Ordinarily, the one-storm rainfall will be greater than the one-day rainfall by about 50 per cent.

The 24-hour precipitation is the most valuable part of Table 1 for a flood-control engineer. The 24-hour data are missing for a large number of stations. For example, in Colorado, the values are given for only three out of fifteen stations. (See Items 281, 289, and 293.)

Table 8 gives the maximum observed precipitation, "day of 24 hours," for certain stations in Colorado, obtained from the U. S. Weather Bureau, and they may be used to fill in Table 1.

TABLE 8.—OBSERVED RAINFALL FOR VARIOUS PERIODS, IN COLORADO.

Station No.	Station.	Maximum observed precipitation (day of 24 hours).	Station No.	Station.	Maximum observed precipitation (day of 24 hours).
280	Steamboat Springs..	2.7	287	Longs Peak.........	4.8
282	Durango............	2.7	288	Greeley............	3.4
283	Wagon Wheel Gap..	2.6	289	Denver.............	6.5
284	Gunnison	1.6	291	Lake Moraine.......	5.5
285	Marshall Pass.......	1.5	292	Colorado Springs...	4.3
286	Leadville............	4.0	294	Las Animas.........	3.4

The author's discussion is interesting and instructive. It would appear that he might have made some more specific deductions after so much study; but the information is given in convenient form for a reader to make his own deductions.

The author has one paragraph entitled "Determinate Limits", in which he indicates that "it has long been recognized that there may be somewhat definite limits of rainfall intensity, frequency, duration, and variation applicable to given localities", and he adds that "an appreciation of the limited scope of human knowledge compared with the broad fields awaiting exploration deters the expression of opinions among those most competent to speak." In various other paragraphs the author evidently has the idea of "Determinate Limits" in mind. The writer wishes to express the opinion that for both practical and theoretical reasons it is much wiser to assume that there is no such thing as a determinate limit of rainfall intensity for any given locality. An effort will be made to reduce the question of determinate limits to concrete terms, and then some of the facts will be given upon which the writer has based his opinion.

According to a formula[36] introduced by the writer, the flood discharge of a stream varies directly with the precipitation in the storm that produces it, when the conditions are favorable for a heavy run-off, and when seepage can be practically neglected. In the United States, east of the Mississippi River seepage can be generally neglected if only the largest floods are being considered. In the arid West seepage is more of a factor, but even there, in the larger storms, it seems to be almost negligible. In any case the relation between precipitation and run-off, or discharge, is such that if there is a determinate limit for the one there is a determinate limit for the other; and, similarly, if there is no determinate limit.

---

[36] Meyer's "Elements of Hydrology," Second Edition, p. 378.

The relation between precipitation and run-off is further confirmed by a study[37] made by F. G. Switzer, Assoc. M. Am. Soc. C. E., and Mr. H. G. Miller.

The Foster method of applying the Pearsonian theory of probabilities, as described[38] by H. Alden Foster, Assoc. M. Am. Soc. C. E., has been adopted by a number of engineers. In using the Foster method for floods, the hydrologist must decide upon the type of curve to use for any particular set of observations. For all practical purposes the discussion may be limited to Type I and Type III. Type I has a definite upper limit, or determinate limit. Type III has no definite upper limit; or, in other words, the Type III probability curve is asymptotic to the X-axis, as is the case with the probability curve in the theory of least squares.

The question of determinate limits is thus closely related to the question of the relative merits of Type I and Type III curves. No engineer can design flood-control works with the meager data generally found in the United States, without basing his design on probabilities. The writer believes that the Foster method will come into wider use by engineers when its possibilities and limitations become better understood.

The question of determinate limits, whether decided one way or the other, will not have an unreasonable effect on engineering design. No matter what the long-time probabilities may be, there is always a determinate economic limit to the amount of money that may wisely be spent on an engineering project. It would not be wise to build levees to protect against a probable 1 000 000-year flood, any more than it would be wise to build houses in Kansas to protect against the worst tornado to be expected once in 1 000 000 years. It is essential that the engineer should have a definite conception of the degree of safety that he is attaining, and that there should be full value, for money spent, in safety, or other desirable features.

To illustrate the practical difference between determinate limits, and no determinate limits, attention is invited to the following figures which are taken from computations made for Bear River, Collinston, Utah:

	Cubic feet per second.
Type I Curve; Determinate Limit:	
10-year flood	10 000
100-year flood	12 000
10 000-year flood	15 000
Type III Curve; No Determinate Limit:	
10-year flood	10 000
100-year flood	14 000
10 000-year flood	22 000

---

[37] "Floods," by F. G. Switzer, Assoc. M. Am. Soc. C. E., and H. G. Miller, *Bulletin*, Eng. Experiment Station, Cornell Univ., Ithaca, N. Y.

[38] "Theoretical Frequency Curves and Their Application to Engineering Problems," by H. Alden Foster, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

Attention is invited to the fact that for the Type I curve the 10 000-year flood is only 50% greater than the 10-year flood. This is a small spread, and would indicate that the river is very regular in its habits.

This illustration is given in order to help visualize the problem. If it is assumed that there is a determinate limit, this would imply a greater degree of regularity in the flood habits of rivers than seems reasonable to the writer. On the other hand, even if there is no determinate limit to rainfall or floods, there will still be a definite limit for economic engineering design.

If the Type III curve is adopted for the Bear River, it would still appear to be a reasonably steady stream, and quite satisfactory from the standpoint of engineering design. The writer has had occasion to use the Foster method with a number of streams of different sizes, and it is thought that the Bear River may be taken as fairly typical of probability characteristics.

The following is taken from the writer's notebook: The maximum flood on the Susquehanna River, at Williamsport, Pa., between 1895 and 1914, was 141 000 cu. ft. per sec. The 1889 flood was 350 000 cu. ft. per sec. Comparing this with the Bear River it could be assumed tentatively that the 1889 flood was about in the 10 000-year class.

The Juniata River, at Huntingdon, Pa., had a maximum discharge between 1896 and 1914 of 37 800 cu. ft. per sec. The 1889 flood was 106 000 cu. ft. per sec., which would seem to be considerably greater than the 10 000-year flood. The writer is of the opinion that neither one of the two 1889 floods was as unusual as indicated.

Another example: The maximum flood on the Arkansas River, at Pueblo, Colo., between 1895 and 1920, was 17 500 cu. ft. per sec. The maximum flood in 1894 was 39 100 cu. ft. per sec. The maximum flood in 1921 was 103 000 cu. ft. per sec. This would indicate that the Arkansas River in Colorado is not in the same class with the Bear River in the neighboring State of Utah. In 1920, the 1894 flood might have been considered near the limit. Then, how should the 1921 flood be classed?

Consider now some of the author's figures for rainfall. In Fig. 5 is represented the maximum 24-hour rainfall in the United States. In the Eastern and Central United States the isohyetals seem to be fairly well smoothed out so as to represent average conditions, except for one place in Southeastern Georgia, and another place in Texas. These two "humps" are undoubtedly due to two particular storms, and they not only stand out above anything in the vicinity, but above any other place in the United States. If the isohyetals in Texas surrounding the high spot are typical, then the one storm in Texas set a high determinate limit for one particular part of Texas. The 23-in. isohyetal is surrounded on three sides by the 8-in. isohyetal.

In Colorado, on the other hand, the isohyetals do not indicate anything out of the ordinary. In Table 1 (Appendix I), the author gives the maximum 1-day rainfall at Pueblo as 2.9 in., based on a 35-year record. According to his remarks on "Climatic Stability", a 35-year record may be considered to be fairly good. Nevertheless, there is evidence[39] that in 1921 there was a fall of about 12 in. of rain in 24 hours, within 6 miles of Pueblo. Furthermore, there is reason to suspect that in the same storm, in one or two small areas, there was an intensity of rain that might be compared to the Texas storm, which set the record for the United States.

---

[39] *Water Supply Paper 487*, U. S. Geological Survey, p. 13.

Fig. 14 is based on the Weather Bureau records of 194 stations, in which the length of records varied from 5 years to 58 years. Isohyetals are based on the highest values of each group. The map is offered as a supplement to Fig. 5, and as illustrating the need for more detailed studies, especially where the topography is irregular.

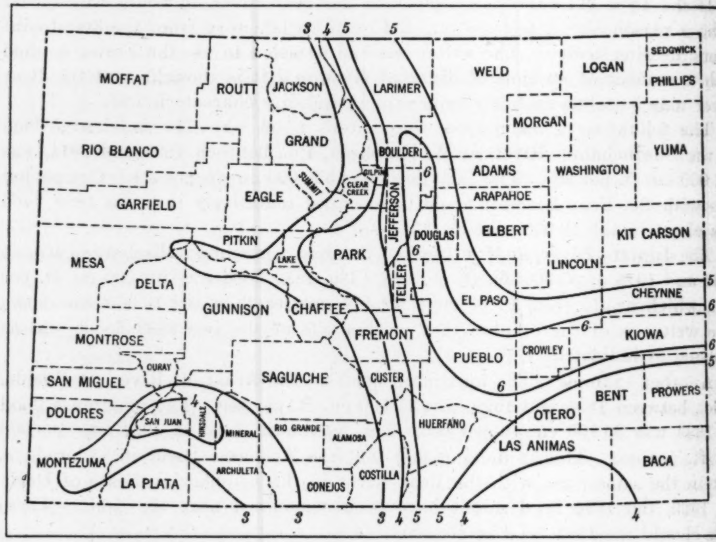

3
4
5
5
MOFFAT
ROUTT
JACKSON
LARIMER
WELD
LOGAN
SEDGWICK
PHILLIPS
MORGAN
RIO BLANCO
GRAND
BOULDER
GILPIN
CLEAR CREEK
SUMMIT
6
ADAMS
WASHINGTON
YUMA
ARAPAHOE
GARFIELD
EAGLE
JEFFERSON
DOUGLAS
ELBERT
KIT CARSON
PITKIN
6
MESA
LAKE
PARK
LINCOLN
DELTA
TELLER
5
CHEYNNE
EL PASO
6
GUNNISON
CHAFFEE
6
KIOWA
MONTROSE
FREMONT
6
5
PUEBLO
CROWLEY
OURAY
SAN MIGUEL
SAGUACHE
CUSTER
6
PROWERS
BENT
4
DOLORES
OTERO
SAN JUAN
HINSDALE
MINERAL
RIO GRANDE
ALAMOSA
HUERFANO
LAS ANIMAS
MONTEZUMA
ARCHULETA
BACA
LA PLATA
CONEJOS
COSTILLA
3
3
3
4
5
5
4
4

FIG. 14.—MAXIMUM 1-DAY RAINFALL, FOR COLORADO, IN INCHES.

"Cloudbursts" should be considered in any detailed study of Colorado weather conditions. It happens that none of the Colorado weather stations has experienced the most severe type of cloudburst, and this should be considered in using the isohyetal map. Largely on account of the topography, storm rainfall in Colorado is much more irregularly distributed than in the Eastern States.

There is need for further research and study along the lines of this discussion. In Mr. Foster's paper previously referred to, he states that "if in doubt as to which type curve to adopt, Type III may be selected as giving results on the safe side". The writer concurs. He would not adopt a Type I curve, "Determinate Limit," for any river with uncontrolled flow, unless he had sufficient evidence to controvert most of his previous experience along this line.

GEORGE C. DANFORTH,[40] M. AM. SOC. C. E. (by letter).—Among points which stand out in this paper are: The great value of the condensed data in

---

[40] Hydr. Engr., Augusta, Me.

Table 1 (Appendix I); a feeling that its value could be measurably increased by the inclusion of certain additional data; and the fact that uncertainties in connection with some of the place names bring out the importance of including the usual geographic co-ordinates for each station.

Incidentally, it seems desirable to express an appreciation of the value of "World Weather Records",[41] which is the source of the major part of the data listed in Table 1 (Appendix I). In this, stations are listed alphabetically by countries, and monthly and yearly values of precipitation, temperature, and pressure are given in the unit used in the original record. While there may be some question as to the advisability of abandoning the 10° squares of latitude and longitude of the Réseau Mondial, an index on that system is appended.

In Table 1 (Appendix I) the monthly precipitation is given for the "normal year", that is, the monthly means for the period of the record, and the stations are arranged in a novel and exceedingly useful form for the study of variations over large areas. While the table is valuable as it stands, it is felt that the engineer interested in run-off would greatly appreciate the inclusion of two additional "years" in the record, namely, monthly precipitation for an average year, and monthly precipitation for a minimum year.

In the United States the excellent data of the Geological Survey leaves little reason for attempting to derive a run-off from rainfall, but the greater part of the world has no available record of stream flow, and rainfall records must be used in preliminary estimates of available power. From this point of view it is felt that records for the average and minimum years are of greater importance than the record for the "normal year."

The importance of the minimum year record will be obvious. The need of figures for the average year (a year of the record the total of which equals or approximates the mean yearly total) is due to the fact that the "normal" year given in Table 1 (Appendix I) is fictitious; it probably never has occurred, and never will actually occur. The method of obtaining the "normal" year is one that smooths out the high spots. The fact that, in a given locality, six or eight months are dry, may not appear in its figures unless the dry months are the same year after year; this is rarely the case in fact. To illustrate the difference in the types of year under discussion, the monthly rainfall for two stations in Brazil is given in Table 9 for the "normal", average, and minimum year of the record. Alto da Serra is in the State of São Paulo, on the coastal side of the Serra do Mar, 2 625 ft. (800 m.) above sea level, latitude 23°47′S. It is in the path of the southeast trade winds, has a sub-tropical climate, and a high rainfall. Quixeramobim, in the State of Ceará, is in the interior of the dry northeast at an elevation of 679 ft. (207 m.), above sea level, latitude 5°16′ S. Its climate is tropical. These data are taken from "World Weather Records."

The stations in Table 1 (Appendix I) seem to be well chosen to give a general idea of climatic conditions. One exception to this may be noted in the case of Brazil.[42] No station is listed to indicate in any way the unusual condition prevailing in the so-called "sertão," the dry interior of the northeast. The coastal stations listed show a normal precipitation and the high rainfall of the Amazon Basin is near; nevertheless, the greater part of the States of

---

[41] Smithsonian Miscellaneous Collections, Vol. 79, 1927.

[42] Table 1 (Appendix I), Stations 375 to 380, inclusive.

Maranhão, Piauhy, Ceará, Rio Grande do Norte, Parahyba, Pernambuco, and Bahia have normally seven dry months or more a year, at which time the country assumes the aspect of a desert. At times, there have been 1-year or 2-year droughts, with heavy loss of life and cattle, and large emigration to other areas, notably during the "Great Drought" of 1877-79 in Ceará. The writer has had occasion to estimate the minimum flow on certain streams in Pernambuco. One appeared to have less than 1 cu. m. per sec. (35.3 sec-ft.) on an area of 2 980 sq. km. (1 852 sq. miles); and on 2 580 sq. km. (1 603 sq. miles) a minimum flow of zero. One station showing these conditions is listed in "World Weather Records", but was omitted from Table 1 (Appendix I). The area is amply covered in reports of the Directoria de Meteorologia de Brasil.

TABLE 9.—MEAN MONTHLY RAINFALL, IN INCHES,* OBSERVED AT TWO STATIONS IN BRAZIL.

Month.	ALTO DA SERRA.			QUIXERAMOBIM.		
	Normal.	Average.	Minimum.	Normal.	Average.	Minimum.
January	16.6	15.4	7.0	3.1	0.8	4.7
February	15.4	20.1	10.3	3.5	5.1	0.3
March	14.4	10.4	14.8	5.7	8.4	1.9
April	12.1	13.0	10.4	4.8	4.3	0.6
May	8.4	10.5	1.1	3.7	2.6	0.6
June	7.7	7.2	0	1.5	2.1	0.2
July	6.7	5.3	7.0	0.8	1.3	1.3
August	7.8	13.9	5.1	0.4	0	0.7
September	10.7	7.0	2.5	0.08	0.1	0.08
October	12.4	13.7	5.4	0.04	0	0
November	14.1	14.1	8.6	0.2	0.3	0
December	14.8	11.0	17.0	1.1	0	0
Total	141.5	141.5	89.3	25.1	25.0	10.3

* Original data in millimeters.

While the addition of monthly temperature and pressure records might not have a value sufficient to balance the loss of compactness, it is felt that the mean annual temperature and pressure would be extremely useful. The temperature is of considerable interest to both the engineer and climatologist. The pressure is of interest in relation to the world belts of high and low pressure, such as the general high pressure (30.0 to 30.1 in.) of 30° latitude, falling off to 29.8° in the equatorial belt and toward either pole (29.9 at 60° N. and 29.3 at 60°S.)[43]

The value of a meteorological record depends much on ability to locate the station with reference to the coast line, mountain ranges, and prevailing winds. At many stations the exact location becomes of importance. For example, Alto da Serra (Station 376) in the State of São Paulo, Brazil, faces the sea near the top of the coastal range, intercepting rainfall from the prevalent southeast trades with resulting heavy precipitation, while the plateau, about six miles inland, is relatively dry.

---

[43] "World Weather," by H. H. Clayton, p. 29.

A casual examination of the remainder of Table 1 (Appendix I) shows that for some stations in Africa the country is given; for others, merely, "E. Africa", "W. Africa," etc. Certain stations in Java, Sumatra, and Timor are listed rather vaguely as "East Indies." The station listed as "Pará, near Belem, Brazil" is decidedly uncertain as to location. Belem is the city which has been called Pará erroneously on some English and American maps in the same way that Recife has been called Pernambuco, and Fortaleza has been called Ceará, using in each case the name of the State for its capital city. Belem is about 100 miles up the Amazon (this southern branch of the delta is actually the Rio Pará). There seems no way of determining from data at hand whether this station is 100 or more miles inland, or actually on the coast at Salinas, a third-class station listed by the Directoria de Meteorologia. In addition to a possible confusion resulting from errors or vagueness in place names, may be noted: (1) The possible duplication of names in the same country; (2) the fact that many towns and cities have changed their names (a common occurrence in South America); and (3) that many places have two names, one only appearing on this or that map. If the co-ordinates were given, these sources of uncertainty would be eliminated.

The statement is made that the greatest rainfall intensity occurs in subtropical and not in equatorial regions. While many of the stations in Table 1 (Appendix I) show no data on daily or hourly maximums, it appears to the writer that the high records are found rather generally in tropical areas. Table 1 (Appendix I) lists ten stations with a mean annual rainfall of more than 200 in. Of these, eight are tropical; the Formosa Station (494) is just inside or just outside the tropics (the name does not appear on available maps), with an elevation of 1 378 ft. The highest record listed is that of Cherrapunji (Station 515), in the Khasi Hills of Assam (Elevation 4 309). Its climate is sub-tropical (latitude 25° 16′ N.) and the mean rainfall given is 456 in., approximately one-half of which falls in June and July, and with a 24-hour precipitation of 40 in. (June 12, 1876). Similar conditions exist on the Western Ghats, a mountain barrier to the southwest monsoon, where precipitation increases with altitude to 250 in. at Mahabaleshwar, while the tableland of Deccan to the east is subject to drought. Baguio (Station 492), in Northern Luzon, with a yearly average of 182.0 in. and a daily maximum of 46.0 in. (July 15, 1911), and with an elevation of 4 959 ft. (latitude about 17° N.), is on a similar mountain barrier to the same monsoon. The wind is from the southeast, but both here and in India the movement during the northern summer is from the sea toward the heated land area of Southeastern Asia.

Of fourteen stations listed with a mean annual rainfall between 150 and 200 in., eleven are tropical and three are temperate, the latter with elevations between 1 100 and 4 400 ft. The highest record shown for North America and South America is that at Buenaventura, on the Choco Coast of Colombia (Station 346), with an average of 280.6 in. Mr. Leo Miller, of the American Museum of Natural History, is authority for the statement that the average rainfall on the San Juan River Basin just north of Buenaventura (latitude about 3° N.), is more than 400 in.

Of these twenty-five high records, twenty are definitely tropical, two are either tropical or slightly over the border, and all five exceptions have elevations between 1 000 and 4 400 ft. It seems probable, however, that the location of these high records in the tropics is due to the fact that monsoon or trade winds moving over a wide expanse of sea are commonly found in tropical areas. When these winds of high humidity meet a physical obstruction, such as the Serra do Mar in Brazil, the hill country in India, or the elevations in Hawaii in the path of the northeast trades, forcing moisture-laden air to cooler altitudes, heavy precipitation occurs, the opposite side of the obstruction being comparatively dry. The forcing of moisture-laden air to cooler altitudes with resulting precipitation may also be a result of atmospheric conditions, such as the meeting of southeast and northeast trade winds on the Lower Amazon.

In discussing altitude and precipitation A. J. Henry, Meteorologist, U. S. Weather Bureau, stated,[44] "The most favorable conditions for an increase in precipitation with increase in altitude, *viz.*, saturated air and relatively high temperature, are found in the tropics." The word, "tropics," in this case presumably refers to the area between 23° 27′ north and south latitude, which is the geographic definition, and the writer has used it with that meaning. From a climatic point of view, however, it seems reasonable to consider it as lying between the two high pressure belts at about 30°, which separate the trade wind areas from the west to east cyclonic movements of the temperate zones. While the equatorial rain belt oscillates roughly between 20° N. and 20° S., the 30° limit would include also the monsoon areas and all the high rainfall records except in the mountains of Wales.

Evaporation.—The evaporation data given in Table 2 are of some interest, together with the inference that they are given for use as indices of evaporation from a free water surface. This table shows for only a small percentage of the stations listed whether the record was from a land or a floating pan and in no instance is information given concerning the size of the pan and the conditions under which it was exposed.

Some confusion still exists (as a perusal of some recent textbooks will prove), concerning the relation between the actual evaporation from open water and that obtained from floating and land pans. Much valuable information on this subject is available in papers presented before the Society, particularly in the discussions on the paper by the late Edwin Duryea, Jr., and H. L. Haehl, Members, Am. Soc. C. E., entitled "A Study of the Depth of Annual Evaporation from Lake Conchos, Mexico,"[45] and in the paper on "Evaporation on United States Reclamation Projects," by Ivan E. Houk, M. Am. Soc. C. E.,[46] and the discussions thereon. The writer has reached the conclusion, from these data, and from his own experiments, that a floating pan properly installed and of such depth that the temperature of the water in the pan is the same as that in the reservoir, will approximate the evaporation from the reservoir. He has also reached the conclusion that the land pan is useless without definite information as to its size and method of exposure, and that its records are of doubtful value unless a coefficient has been obtained by comparison with a properly installed floating pan at the same site. If monthly records are to be used, the coefficient must be obtained from monthly comparisons as it varies decidedly throughout the year.

---

[44] *Monthly Weather Review*, January, 1919, p. 34.

[45] *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1829, *et seq.*

[46] *Loc. cit.*, Vol. 90 (1927), p. 266.

C. S. JARVIS, M. AM. SOC. C. E. (by letter).—The generous response in the way of discussion and constructive criticism from members of the engineering and other scientific professions has brought forth essential items or phases of the subject which no single viewpoint could encompass.

In spite of earnest endeavors to condense data to the utmost limit compatible with reasonable use, the tabulation was regarded by some reviewers as entirely too voluminous for reproduction. It is evident from occasional comments within the discussions, and likewise freely admitted by the writer, that improvement might have resulted from extending the number of stations. The 820 meteorological records and subsequent interpolations listed in Table 1 (Appendix I) were selected from 10 000 or more consulted. In an effort to be consistent, the number of representative stations chosen from among highly developed and populous countries was stringently reduced from those available, while careful gleaning of all fragments of data was required in undeveloped regions, such as the deep interior of Africa, in order that any semblance of a comprehensive view could be provided.

The great danger of "deployment in a thin line to traverse a wide front" is that the individual responsibility is thereby increased, and the danger of wide gaps intensified. Continuing the analogy, the relatively few rainfall stations selected at wide intervals to represent groups in adjacent territory, failed in some instances to reflect the entire range of behavior or precipitation patterns. However, doubling or trebling the volume of data in Table 1 (Appendix I) would not have eliminated this inherent weakness entirely, due to sparsity of numbers, and would have rendered it somewhat unwieldy without commensurate returns in the quality of information.

Mr. Bliss evidently recognizes the preliminary steps leading up to the design and formulation of projects for flood control; namely, evaluation of rainfall intensities, duration, and intervals between probable recurrences. These, with other influences, are, after all, translated by natural phenomena into such stern realities as river gauge heights and corresponding velocities, discharge, and destructive dynamic effect.

Major Somervell points out the errors into which one might be led through neglect of the local station records. No method has yet been devised to supplant the use of all available data for a given problem. The comparatively long and complete record of Washington, D. C., so nearly identical with those of Baltimore, Md., and intervening stations, was chosen to represent the coastal plain section; while the Elkins, W. Va., meteorological data seemed to typify behavior of the mountainous region from which the head-waters of the Potomac and Monongahela Rivers flow. Neither was intended to represent an average value for the water-shed, nor an extreme of range. It is only by coincidence that the average annual rainfall determined by stream survey investigations for the Potomac River Basin should agree so closely with the mean published for the National Capital.

It is not at all surprising that extremes of annual rainfall at other stations should go beyond the range of the Elkins and Washington records. Bayard, W. Va., at Elevation 2 700, and with annual mean of 53.99 in. for 22 years of record extending intermittently to include 1929, should normally record a correspondingly higher maximum than that for Washington in nearly the ratio of their respective means, 40.5 to 53.99 in., in a given period. By proportion, 40.5 : 53.99 : : 61.3 : 81.7, showing that the maximum of 89.01 in. recorded for Bayard during 1926 is not far beyond the normally expected range for a century.

The minimum of 12.71 in. for 1895 at Western Port, Md., is probably in error, as is made quite evident by a comparison with either the few near-by stations, or the several in the surrounding country listed in Table 10. Other circumstantial evidence against the reported rainfall during 1895 at Western Port accrues from the fact that this was the first full year of record, and was subject to the disadvantage of local inexperience regarding standard methods; the recurrence of four annual rainfall depths, next in order as minima during a 35-year record, ranging between 26 and 27 in.; and the surprising deficiencies, month by month throughout most of the year, as compared with the near-by stations of Cumberland, Boettcherville, and Burlington (six-month record only), which occupy comparable positions on the terrain, although at somewhat less elevation.

Whether the Western Port minimum annual precipitation of 12.71 in. is regarded as an error in observation, or only a local freak of a sub-normal rainfall, it cannot be regarded as representative over any considerable section of the Potomac River water-shed; it would be more consistent to base judgment on a group of meteorological stations, or selected ones which represent their average behavior.

The principal value of such summaries as comprise Table 1 (Appendix I) is in furnishing a background against which the detailed data of local stations may be more broadly interpreted. No claim has been made that such a summary will supplant local information; instead, it calls for all the information available, and assists in utilizing fragmentary data. A broad view of a field of research should precede the detailed study.

Mr. Shuman brings forth in attractive form the claims for periodicity of rainfall variations with relation to sunspot numbers. No one can doubt that series of years may differ in total precipitation; but the abrupt manner of changing from one extreme to another supports the impression that a portion of the normal rainfall that was temporarily withheld was later released; or a reserve somewhat depleted by unusual expenditures must await recuperation before resuming normal habits.

The comment from Mr. Shuman's discussion to the effect that, "if too large an area is taken, the data at one end or part of the district may neutralize those of another section, and one may secure no recognizable cycles," displays the weakness of the claims for sunspot influence. It reduces the coincidences to local happenings entirely, as the writer sees the problem. Ever

TABLE 10.—Comparison of Annual Rainfall Depths*

Stations	Elevation above sea, in feet	Years of Record		Mean annual rainfall	Observed Rainfall During Designated Years										
		Number	Last year included		1890	1891	1892	1893	1894	1895	1896	1897	1898	1899	1900
Morgantown, W. Va	1 250	45	1920	43.73	69.54	54.65	38.59	32.43	31.05	28.15	41.56	42.77	52.12	42.53	39.43
Rowlesburg, W. Va	1 375	36	1920	47.54	56.46	49.05	39.91	36.26	45.34	31.79	41.09	49.34	44.33	47.64	36.60
Buckhannon, W. Va	1 411	24	1920	50.12	67.55	57.29	44.75	41.88	38.07	35.19	52 85	50.51	61.14	......	......
Burlington, W. Va	875	29	1924	35.52	......	......	......	......	27.75	28+ †	42.22	32.95	40.78	36.58	27.67
Martinsburg, W. Va	435	29	1920	37.59	......	......	29.47	38.11	33.66	28.98	31.78	35.34	38.60	33.76	30.65
Harper's Ferry, W. Va	244	31	1920	36.67	36.60	31.83	33.93	30.65	30.32	29.80	34.33	43.25	44.48	39.93	34.98
Western Port, Md	1 000	35	1929	34.60	......	......	......	......	......	12.71‡	37.18	32.61	42.90	36.84	30.64
Deer Park, Md	2 457	24	1918	44.85	......	......	......	......	......	29.50	49.81	44.74	53.12	37 .92	36.21
Oakland, Md	2 461	20	1920	45.66	......	......	......	43.97	44.86	40.66	......	......	......	......	......
Boettcherville, Md	780	14	1905	35.31	......	......	34.95	37.85	32.60	28.20	43.20	35.30	38.06	28.69	28.43
Cumberland, Md	623	56	1929	35.08	52.42	46.29	34.66	30.04	28.10	23.03	37.54	35.92	41.14	34.18	28.85
Grantsville, Md	2 851	26	1920	42.48	......	......	......	......	......	34.16	50.75	45.92	55.52	39.86	36.00
Hancock, Md	455	10	1905	35.56	......	......	......	......	......	28.21	35.19	......	39.37	33.16	30.30
Sharpsburg, Md. (localizing about Western Port, Md.)	420	10	1904	36.49	......	......	......	......	......	26.76	33.18	38.57	39.94	36.56	29.32
Boettcherville, Md	780	14	1905	35.31	......	......	34.95	37.85	32.60	28.20	43.20	35.30	38.06	28.69	28.43
Cumberland, Md	623	56	1929	35.08	52.42	46.29	34.66	30.04	28.10	23.03	37.54	35.92	41.14	34.18	28.85
Western Port, Md	1 000	35	1929	34.60	......	......	......	......	......	12.71‡	37.18	32.61	42.90	36.84	30.64
Burlington, W. Va	875	29	1924	35.52	......	......	......	......	27.75	28+	42.22	32.95	40.78	36.58	27.67

* With such consistent relations between the records at Western Port, Md., and neighboring stations, in all other years, it appears that the 1895 record is in error.

† At Burlington, January to June, 1895, total depth of 14.91 in.; no record for remainder of 1895; probably annual rainfall for entire year exceeded 28 in.

‡ First year of record; probably in error; rainfall for the year may have exceeded 20 in.

since reliable records were initiated, there have been areas of either excessive or deficient rainfall reported; and these again have years of above normal or below normal deliveries for their respective localities. The Eastern States suffered from drought during the summer of 1930 while various other sections of the country had unusually heavy precipitation, and floods in various parts of Europe were wreaking great destruction.

If an investigator has the right to bring station records into apparent agreement of phase by assigning lags of up to ten years, then it is freely admitted that the crests and troughs of rainfall graphs may be made to coincide, for occasional short sections, with the various cycles devised, while other stations are in opposition. Why not accept the conclusions promulgated by careful observers after diligent research—that sunspot increase was accompanied or followed by increase of precipitation on about one-third of the land surface, and by a decrease on nearly an equal area; while, for the remaining third of the earth's surface, no conclusive data were available. The presumption is that either claim may be proved or disproved by nearly one-half the data, which to the writer indicates that there is "no direct relation traceable between rainfall and sunspot periodicity." If such a relation exists, as a number of reputable scientists earnestly believe and seek to establish, it is either too elusive or too thoroughly masked by local influences for definite identification at this time.

Although the pluviographs in Fig. 2 depict the behavior of only nine widely separated stations, the preliminary studies included platting of more than one hundred of the longest records chosen from all over the earth's surface, and there were nearly as many oppositions as agreements among the crests and troughs, in comparison with sunspot numbers, whether the latter are platted inverted or above the base line and whether annual rainfall for consecutive years or the progressive two-year means for the entire periods are platted.

The profession is indebted to Mr. Shuman for the clear exposition of his views on periodicity.

Mr. Bogert has pointed out how the value of the tabulation might be increased by designation of the time of occurrence for maxima and minima, both yearly and monthly. Such information could be used to advantage; but obviously it calls for another tabulation containing such supplemental data. An attempt was made to record such dates in the original compilation; but the encumbrance entailed thereby seemed to outweigh any immediate advantage.

Professor Powell has contributed interesting and valuable views on the subject of periodicity, with reference to the recurrence of maxima or minima of a given order. The writer recognizes the alternation of sub-normal, medium, or excessive rainfall years, either singly or in groups, as they happen to occur. A season of excessive precipitation for one locality occurs simultaneously with droughts in other districts. It seems to be largely a question of routing; a large number of natural forces and elements jointly determine the path traversed by the atmospheric currents on which procurement, transportation, and delivery of moisture depend. To extend the analogy with other supply problems, the first few thousand feet of atmospheric strata above the surface of the globe, whether land or sea, high or low in either elevation or latitude, represent the loaded vehicles; the counter-currents at higher levels represent "the return of the empties", or the vehicles of reduced capacity.

The foregoing view is supported, if not established, by the physical data of Table 3, which shows that, with normal distribution or gradation of temperature with altitude, and the corresponding moisture capacity, the lower 5 000-ft. depth of atmospheric strata, if at saturation, contains considerably more moisture subject to precipitation than all the combined layers above. That relation holds, no matter whether the reckoning begins at sea level, or from mountains of any known height.

The forces which are accountable for tidal phenomena and ocean currents should extend a kindred influence on atmospheric movement. A tidal range of about 3 ft. occurs in mid-ocean; a range of 7 ft. is not unusual along extensive coast lines; more than twice this amount, occasionally quadruple, occurs under special conditions. Usual ocean tidal phenomena correspond to ordinary barometric pressure variations, measured by about ½ in. of mercury column at official meteorological stations, if due account is taken of the higher mobility of gases and the greater weight of water, that is, a 34-ft. column balancing the entire atmospheric pressure over an equal area at sea level.

For the extremes of observed readings, about 27.6 and 31.4 in. at the same datum, showing a total variation of more than 12%, the analogous ocean tidal range is 30 ft., or more, which may represent a considerably greater proportion of the effective offshore depth of water than the 12% range of barometric pressure noted in the foregoing remarks.

The alternation of high and low tides throughout the day seems to have its counterpart in atmospheric movements producing recurrent showers, such as those so often observed near noon or mid-afternoon, depending somewhat on latitude and the sun's declination. In desert regions, the increased convectional circulation during midday is readily traceable by the numerous whirlwind eddies, bearing aloft bits of leaves or other light substances, in a veil of dust, and displaying the main characteristics of cyclones in miniature, except that there is seldom attendant precipitation. When the desert heat has mounted under the blazing sunshine of successive cloudless days, and has approached the limit of human endurance, the most positive and pronounced relief may follow a violent thunder shower or dust storm. The first warning is likely to be a dust cloud on the far horizon, riven by lightning flashes; then a dull roar, increasing momentarily and punctuated by occasional thunder. In contrast, the prevailing calm and stillness are almost ominous until the moving curtain is within perhaps less than a mile of the observer, when he must suddenly combat fitful wind and cross-currents, followed by a murky cloud raised from the sweeping of the desert floor; then, infrequently, a brief deluge of rain, fiercely driven or occasionally interspersed with hail; and, finally, a few moments after the drenching, a cool, cleansed, and freshened atmosphere rapidly resumes the calm such as preceded the storm.

If the analogy continues to hold between atmospheric and ocean tides, there should be a fortnightly recurrence of unusually energetic circulation of the atmosphere, corresponding to spring tides at seaports. Whether or not precipitation accompanies such disturbances, depends largely on relative humidity and the temperatures that prevail locally; in short, on the climatological habits. A systematic examination of prominent long-record meteorological data, day by day throughout any month or series of months, ordinarily discloses pronounced periods of maxima in wind movement, and, less frequently, in rainfall, occurring at fortnightly intervals or less. Secondary disturbances interspersed probably account for some of the occurrences.

From a consideration of evaporation rates from either free water surface, wet ground, or dense foliage, together with atmospheric moisture capacity, relief by condensation at intervals of a month or less may be regarded as unavoidable at some place along the path traversed, unless intermixed with air currents deficient in humidity. Whether the evaporation rate is the usual 4 in., or more, for months of moderate temperature, or less than 1 in. for a winter month in temperate zones, the corresponding moisture capacity of the entire atmospheric column would not be greatly in excess thereof for average prevailing temperatures. Under such conditions it appears that the initial steps in precipitation would begin at the nearest barrier, whether coast line, mountain range, or mass of dense, cold air obstructing the path of the warmer, moisture-laden currents.

After several hours of intense sunshine, without the moderating effect of free water for evaporation, the upward displacement of warm air to re-establish equilibrium doubtless produces an effect comparable with high tide in sea movements, and thereby affects rainfall distribution nearly as definitely as do the physical obstructions in mountainous regions.

A large number of cross-sections across mountain ranges discloses a fairly definite relation with respect to slopes. It appears that the abrupt changes in gradient also mark pronounced variations in rainfall. The top of the steepest slope, the highest point from which the entire lower slopes can be viewed (known in tactical problems as the military crest), seems to receive the greatest precipitation; and the actual summit may record a considerable reduction in amount. The leeward slopes with respect to moisture supply almost invariably receive deficient rainfall because of the increase of moisture capacity with temperature during the descent.

Professor Cox has pointed out such defects in the platting of isohyetals in Fig. 5 as must inevitably occur, in local sections, if only a few selected stations are utilized. On this account, General Note No. 3 of Fig. 7 is equally applicable to related Figs. 5 and 6, and was so intended, reading as follows: "This map is subject to revision by addition of data covering local variations not considered in Table 1".

It should be observed that the data from Table 1 (Appendix I), for Iowa stations, with one exception, terminated with 1920. Up to that time, the infrequent, sporadic occurrences of more than 7 in. in 24 hours, were so out of line with the usual habits that it appeared inadvisable to let those isolated data govern the platting in opposition to the general habits. Without exception such rainfall was the result of thunder showers during summer or adjacent months and confined to restricted areas entirely away from the longest established and best equipped stations. Such problems demonstrate the importance of introducing the dimension or element of periodicity for interpreting and classifying observations of natural phenomena, as illustrated in Table 11 for annual rainfall.

From the new data contributed by the discussion, it appears that either an unusual number of intense 24-hour rainfalls have occurred in Iowa during the past decade, or else the list of stations must have been extended recently.

Mr. Danforth expresses an appreciation of "World Weather Records" in which the writer concurs, as far as the arrangement, convenience, and value of the data are concerned. This volume was published during the writer's quest for basic information, and includes 355 precipitation records, of which 251 were incorporated in Table 1 (Appendix I), in many instances supplemented by fragmentary data from other sources, or by extensions of summations so that the new means and extremes cover the entire period of available record.

The latitude and longitude co-ordinates were retained for purposes of identification of obscure or remote stations during the process of platting; but after the routing and the plotting of all stations on Fig. 7, the use of an entire line across two pages for such co-ordinates seemed to be out of proportion to the service that could be rendered thereby. Whenever the question of exact location enters into the problem, the length of record, the altitude of a station, and the rainfall patterns into which both the annual and the monthly means and extremes readily classify, should identify the particular station listed, even if there are others of the same designation, as in the case of two named Alto da Serra, in Brazil.

The use of such familiar names as Pernambuco and Para to indicate the principal ports of States bearing the same names, although technically incorrect, seems to be almost universal in news items, and extends to such standard volumes as "The Climate of the Continents",[47] by W. G. Kendrew.

Colonel Pettis has given evidence of broad experience and understanding concerning flood-flow phenomena and their relation to rainfall. The supplemental data for 24-hour periods at ten of the Colorado stations are gratefully acknowledged.

The question of determinate limits as used in the original paper was not intended to go beyond certain considerations, either axiomatic, logical, or demonstrable in the light of established facts. However, the discussion of typical curves derived from a study of probabilities is both interesting and instructive.

Conclusions.—For purposes of classification, it appears logical to compare rainfall distribution, either seasonal or annual, with normal evaporation from free water surfaces for the same periods. Where the potential losses from evaporation are approximately equal to the normal precipitation, the needs of ordinary crops on loamy or other fine-textured soils are abundantly supplied without recourse to artificial methods of application during average years.

---

[47] Oxford, Clarendon Press, 1922.

Where rainfall is habitually greatly in excess, the over-supply is manifested in the marshy character of both the soil and the vegetation, unless either surface or sub-soil drainage is unusually well developed. The regulating effect on run-off thereby produced is generally accentuated by the attendant habits of frequent gentle precipitation.

In contrast with the superabundant rainfall is the deficient rainfall, amounting to decidedly less than normal evaporation depths from free water surfaces, and ranging anywhere below, say, two-thirds of that amount. In a general way, the rainfall habits are recorded by both the surviving vegetation and the soil structures developed under varying degrees of aridity. The limits of plant rootage and moisture percolation, saturation, oxidation, leaching, and deposition are well marked by horizontal stratification and gradation of coloring, or by the widespread calcareous accretion in subsoil known as caliche or hardpan. The latter conserves to the utmost the sparse moisture supply, and hastens saturation or water-logging when it is supplemented by artificial means or by seasons of abnormally high rainfall.

With due regard to the foregoing definitions, the 15-in. average annual precipitation prevalent over extensive areas of Alaska, Canada, Greenland, Northern Europe, Siberia, and parts of the high Andes Mountain Range is comparable with approximately a 40-in. depth over the Middle Atlantic coastal plain, or with nearly double this rainfall in tropical lowlands; and, in each instance, the annual precipitation and evaporation should nearly balance. As progressive increase in altitude along a high mountain slope nearly reproduces the climatological and soil conditions of higher latitudes, the elevations above sea level for meteorological stations supply some of the primary factors in the study of rainfall and run-off; also, in the classification as to aridity or humidity of climate as determined by requirements for plant growth. For simplicity, the three principal classes previously defined may be regarded as those with deficient, moderate, or optimum (in technical terms for "ideal" or "most favorable" conditions), and, finally, excessive precipitation.

The temperate days and cool nights with heavy dew, so prevalent in the uplands as well as in high latitudes, reduce the demands for moisture measured as rainfall. This fact plus the advantage of strategic position, with priority privileges, so to speak, accorded to high mountain slopes, re-acts to make the uplands relatively immune to drought, while the lower interior valleys and plains are the first to suffer depletion, and the last to be served anew. High mountain peaks are swathed in fog, mist, or light rain, usually for hours both before and after the downpour that reaches the lowlands. The greater heights through which the droplets fall to reach the valleys afford opportunity for increased diameter of raindrops, velocity of descent, amount of electric potential induced thereby, further coalescence through attraction of opposite charges; and, finally, the lateral displacement of condensed moisture from its normal path of descent by cross-winds, producing an effect similar to the folding of curtain draperies, and resulting in multiplied intensity of rainfall in some parts of the storm area at the expense of depletion for others, located to windward.

TABLE 11.—EXTREME AND MEANS OF ANNUAL PRECIPITATION FOR VARIOUS PERIODS
(Based on Analysis of Existing Records Selected at Random for Length and Geographical Distribution)

Serial number from Table 1 (Appendix I)	Station	Years of record	EXPRESSED, IN INCHES							EXPRESSED, IN PERCENTAGE						
			Minima			Mean	Maxima			Minima			Mean	Maxima		
			100 years	50 years	10 years		10 years	50 years	100 years	100 years	50 years	10 years		10 years	50 years	100 years
15	Burlington, Vt.	93	21	24	28	32.6	36	43	49	64	74	86	100	111	132	150
23	Boston, Mass.	110	27	30	34	43.5	52	60	67	62	69	78	100	120	138	154
39-*A*	New York, N. Y.	101	29	31	34	42.1	48	55	59	69	74	81	100	114	131	141
43	Albany, N. Y.	101	26	28	30	37.4	42	52	57	69	75	80	100	113	139	153
58	Philadelphia, Pa.	110	29	31	35	42.4	49	55	61	68	73	83	100	116	130	144
69	Pittsburgh, Pa.	91	25	28	30	36.2	41	44	51	69	77	83	100	113	121	141
76	Washington, D. C.	89	19	24	32	40.5	49	58	61	46	59	79	100	121	143	150
97	Charleston, S. C.	136	24	31	33	47.5	62	70	78	51	65	70	100	131	148	165
109	Key West, Fla.	78	21	24	30	37.9	49	56	70	55	63	79	100	129	148	185
119	New Orleans, La.	81	31	39	44	57.0	68	81	86	54	69	77	100	119	142	151
139	Little Rock, Ark.	48	....	31	35	48.2	62	76	....	....	65	73	100	128	158	....
148	St. Louis, Mo.	91	23	26	31	39.7	49	61	69	58	65	78	100	124	154	174
167	Chicago, Ill.	84	22	24	27	33.1	40	44	47	66	73	82	100	121	134	142
171	Cincinnati, Ohio.	93	18	27	30	40.7	49	55	65	44	66	74	100	121	135	160
185	St. Paul, Minn.	90	10	16	21	27.2	35	40	50	37	59	77	100	129	148	184
204	Seattle, Wash.	52	....	22	27	33.6	43	56	....	....	65	80	100	128	167	....
226	San Francisco, Calif.	80	9	10	15	22.3	30	37	39	40	45	67	100	135	166	175
236	San Diego, Calif.	80	3	4	6	9.6	14	19	28	31	42	63	100	146	198	292
258	Sacramento, Calif.	80	8	9	12	18.3	23	30	35	44	49	66	100	126	164	191
273	Salt Lake City, Utah.	55	....	10	12	16.3	21	24	....	....	61	74	100	129	148	....
336	Colon, Panama	57	....	86	103	127.3	154	183	....	....	68	81	100	121	144	....
377	Rio de Janeiro, Brazil.	75	....	29	32	43.2	55	65	....	....	67	74	100	127	151	....
401	Havana, Cuba.	67	....	28	41	48.2	59	65	....	....	58	85	100	123	135	....
416	Sitka, Alaska.	64	....	58	70	84.8	102	140	....	....	69	83	100	121	166	....
468	Sydney, Australia.	85	21	24	35	47.9	63	81	83	44	50	73	100	132	170	174
498	Tokyo, Japan.	45	....	46	52	59.9	77	86	....	....	77	87	100	129	144	....
516	Calcutta, India.	92	39	43	52	64.3	76	89	93	61	67	81	100	118	138	145
521	Madras, India.	108	18	22	34	49.6	70	80	88	36	44	69	100	141	161	177
567	Capetown, South Africa.	87	17	19	21	25.3	31	36	41	67	75	83	100	122	142	162
618	Beirut, Syria.	45	....	23	27	35.5	46	51	....	....	65	76	100	130	144	....
649	Milan, Italy.	161	25	27	30	39.4	49	54	62	64	69	76	100	125	138	158
651-*A*	Padua, Italy.	202	19	21	25	33.8	44	50	62	56	62	74	100	130	148	183
654	Rome, Italy.	143	13	20	28	32.7	41	45	58	40	61	86	100	126	138	177
662	Lyons, France.	80	19	20	24	29.1	35	39	45	65	69	83	100	120	134	155
665-*A*	Paris, France.	214	9	11	14	19.9	25	28	30	45	55	70	100	126	141	151
695	Greenwich, England.	81	16	18	20	24.3	30	34	36	66	74	82	100	123	140	148
748	Warsaw, Poland.	109	15	16	18	21.2	27	31	46	71	75	85	100	128	146	217
753	Leningrad, Russia.	95	12	15	17	19.2	24	27	29	63	78	89	100	125	141	151
797	Helsingfors, Finland.	80	14	15	19	24.4	30	32	34	57	61	78	100	123	132	139
807	Copenhagen, Denmark.	100	14	16	18	22.5	27	29	30	62	71	80	100	120	129	133

Such considerations account in part for the gentle, persistent rainfall of humid areas as opposed to the irregular, fitful, and violent phenomena of arid or semi-arid sections. In terms of stream flow or water supply, there is the discrimination between primary and secondary rights, with their contrasts of dependability. Whether the favored strategic position depends for its advantage on altitude, or on proximity to either the source of atmospheric moisture or its main lanes of traffic, or on the abrupt steepening of gradient to surmount a barrier, the beginnings of condensation merge almost imperceptibly with fog for part of the time.

If the wind movement becomes unusually strong, or if the energy released by the process of condensation is transformed into the kinetic phase to act as a propellant, then the storm may carry over the leeward side of the barrier, whether it is a land form, an underflowing mass of dense atmosphere, or merely the impediment of barren distance, for the relatively brief period of greatest activity and intensity.

The high ratios between successive values of either rainfall or percentage as given in Table 1 (Appendix I), are apparently a fair measure of relative aridity, whether the daily, monthly, or yearly periods are the basis of reckoning. For either moderate or excessive precipitation stations, either the depths of rainfall or the percentages of the mean annual which they represent (as tabulated), make fairly regular ascending series when read from left to right. For stations with deficient rainfall in comparison with potential evaporation from free water surfaces, the series is broken by occasional inversions or reversals. For example, the maximum monthly rainfall may far exceed the minimum annual, instead of being approximately one-half or two-thirds of the value in the next column, as seems to be characteristic of stations with dependable and ample rainfall. Furthermore, for stations of moderate rainfall, a typical 50-year record may show minimum and maximum percentages of about 50 and 150, respectively, or, more often, 60 and 140. For excessive rainfall stations, 70 and 130% may be more nearly representative, while for regions of marked deficiency of rainfall compared with potential evaporation, the range may be 40 and 200%, or, more rarely, nearly 30 and 300 per cent. Moreover, the average 10-year minima and maxima should ordinarily fall almost intermediate between the 100% and the foregoing extremes, although there may be lack of symmetry for some records, and decided departure from arithmetical series for others. The first approximations for 10-year minima and maxima for all the foregoing cases, and, in the same order, therefore, would be 75 and 125; 80 and 120; 85 and 115; 70 and 150; and, finally, 65 and 200%, respectively. Table 11 displays results of further studies along this line. Some of the irregularities may have resulted from varying reliability of either the rainfall data or the results of periodicity analysis.

As may reasonably be expected, the periodicity relations dealing with monthly rainfall display the same general trends and characteristics, but involve higher ratios. A fairly promising field awaits development along the same lines, but will require organized effort and facilities.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

TRANSACTIONS

Paper No. 1773

BALDWIN FILTRATION PLANT, CLEVELAND, OHIO[1]

By J. W. Ellms,[1a] G. W. Hamlin,[2] A. G. Levy,[3] and J. E. A. Linders,[4]
Members, Am. Soc. C. E.

With Discussion by Messrs. S. M. Van Loan, Harry N. Jenks, James H. Herron, E. Sherman Chase, and J. W. Ellms, G. W. Hamlin, A. G. Levy, and J. E. A. Linders.

Synopsis

In providing for the growth of American cities, one of the major problems is that of maintaining an adequate and sanitary water supply. The rapid increase in population in Cleveland, Ohio, during the past two decades has necessitated a careful study of this particular problem. For more than fifteen years, the Engineering Division of the Department of Public Utilities has investigated, planned, and constructed means for providing an ample supply of pure water.

While the location of Cleveland on Lake Erie permits of an ample supply of water for the city proper as well as for a Metropolitan District involving about 500 sq. miles, the need for providing water of the proper quality had not received much consideration until about 1913. The City of Cleveland supplies water to about thirty-five political subdivisions of Cuyahoga County and to several communities outside that county. For the most part, these do not border upon the lake and, therefore, it has been necessary to consider the problem of water supply for Cleveland as one that in reality involved a Metropolitan District.

The object of this paper is to describe a part of the development of a comprehensive plan for supplying this district with pure water. Although

---

[1] Published in February, 1930, *Proceedings*.

[1a] Engr. of Water Purification and Sewage Disposal, Dept. of Public Utilities, Cleveland, Ohio.

[2] Engr. of Constr., Div. of Water and Heat, Dept. of Public Utilities, Cleveland, Ohio.

[3] Engr. of Constr. and Surveys, Div. of Water and Heat, City of Cleveland, Cleveland, Ohio.

[4] Engr. of Design, Dept. of Public Utilities, Div. of Water and Heat, Cleveland, Ohio.

the City had constructed a filtration plant on the west side of the Cuyahoga River, which went into operation in 1918, only about one-half the supply was filtered until the Baldwin Plant was placed in operation in 1925. The water drawn through the City's intakes in Lake Erie is not as grossly polluted as many of the sources of public water supply of larger cities, but the growing population adjacent to the lake in this vicinity and the increased discharge of sewage directly into the lake have produced conditions that demanded an efficient purification system for the public water supply. This paper describes a large filtration plant into which has been introduced the most modern methods in the art of water purification and the largest covered concrete reservoir in the world. The paper has been subdivided as follows: History and Development of the Project; General Description of Flow of Water; Hydraulic Data; Design; Construction; General Equipment; Completion of Baldwin Filtration Plant; and Amount of Contracts and Analysis of Costs.

HISTORY AND DEVELOPMENT OF THE PROJECT

The southern portion of Lake Erie opposite the City of Cleveland is the source of supply for all water furnished by the municipality to the city proper, its suburbs and neighboring villages, as well as to the more distant parts of Cuyahoga County and a small part of Lake County to the east. Because this water is low in turbidity, averaging about 15 parts per million, no attempt was made until late in 1911 to treat the water furnished for consumption, in any way, except by chlorination. At that time, after a more or less prolonged controversy regarding the necessity of filtering the water supply, a decision was reached to start plans for a filtration plant. About the beginning of 1918, the Division Filtration Plant, on Division Avenue, between West 29th and West 53d Streets, with a capacity of 150 000 000 gal. per day, was placed in service, thus giving a section of the City of Cleveland its first supply of filtered water. In general, this plant was capable of supplying only that part of the city and vicinity lying west of about East 55th Street. The remainder of the area supplied continued to be furnished with chlorinated raw water only. However, before the Division Filtration Plant was completed, plans were under way for a new plant which, in conjunction with that at Division Avenue, would be capable of supplying the entire city and its principal suburbs with filtered water.

In line with the usual procedure of locating a filtration plant near the pumping station supplying it with raw water, the original plans for this plant contemplated its construction near the Kirtland Pumping Station, on the lake front, at the foot of East 49th Street. A study of this location, however, indicated that conditions were such as to require pile foundations for all structures. The area available on which a plant could be built was insufficient without a considerable fill being made in the lake, and this, in turn, necessitated the use of a more or less expensive coffer-dam type of construction. Because of the difficulties to be overcome at the lake front site, the Engineers of the Cleveland Water Department gave consideration to the possibility of utilizing some other location.

It became evident about 1907, that the existing Fairmount Reservoir, supplying the low-service district, was becoming inadequate for storage purposes. It was also desired to increase the pressure in this service. Accordingly, during 1908 and 1909, the City purchased land for a new low-service distributing reservoir. This land was located directly east of Fairmount Reservoir and extended from Baldwin Road to East Boulevard. The new land was about 60 ft. higher than the surface of the water in the old reservoir, making possible increased pressures in the area to be served.

The possibility of utilizing the area in the vicinity of Baldwin Reservoir as another site for the new filtration plant was given serious thought. An estimate of cost revealed the fact that, with all factors taken into consideration, the Baldwin site was the cheaper. This finding was concurred in by an independent commission of engineers, composed of Messrs. J. W. Frazier, J. H. Herron, and Robert Hoffman, Members, Am. Soc. C. E., in a report dated June 1, 1920. As a result, the Baldwin Filtration Plant with a capacity of 165 000 000 gal. per day, was located on the Baldwin site, along the north, east, and south sides of Baldwin Reservoir. This location is approximately 3½ miles from Lake Erie.

The area within which the Baldwin Filtration Plant is located is approximately 50 acres. The filtration plant structures occupy approximately 11 acres and the Baldwin Reservoir about 13 acres. Fig. 1 shows the general location of the plant and the direction of flow of water.

The territory which the City of Cleveland will ultimately supply with water is a large metropolitan district destined to cover the entire area of Cuyahoga County and even to spread beyond its borders. The two existing filtration plants, having a total capacity of 315 000 000 gal. per day, will supply all needs until about 1930. By that time a third plant will be required in the eastern half of the county, and about ten years later a fourth in the western half.

General Description of Flow of Water

The raw water for the Baldwin Filtration Plant is drawn from Lake Erie through a steel and concrete crib about 4 miles offshore, and thence through a 9-ft. brick-lined tunnel about 26 000 ft. long to the Kirtland Pumping Station. From there it is pumped through one lock-bar steel and three cast-iron raw-water mains, each 48 in. in diameter and about 7 500 ft. long. These four mains combine into two 60-in. lock-bar steel pipe lines, each approximately 15 000 ft. long, leading to Fairmount Reservoir. The water is then lifted by centrifugal pumps in Fairmount Pumping Station (see Fig. 1) and is forced through two 60-in. cast-iron mains, each approximately 2 100 ft. long, up the rising well in the chemical house of the filtration plant, to the upper pool of the three hydraulic-jump mixing flumes (capacity of each flume, 55 000 000 gal. per day).

Passing through these flumes (Fig. 1), the water flows through a channel approximately 124 ft. long, containing two pairs of under and over-baffles; and thence through Gate-House No. 1, into the south conduit of the coagulating basins. Here, it is distributed through two gate-houses into the four basins themselves. The water enters the coagulation basins at the south end over submerged weirs, travels through them, and leaves at the north end over similar weirs. The water then flows through the control gates and a conduit to the Administration Building, passing on its way through Gate-House No. 6.

The flow of water divides in the Administration Building, supplying the filters as shown in Fig. 1. From the filters, it passes through rate controllers into effluent galleries or small storage reservoirs. There is one gallery under each group of ten filters. The water flows from each of the galleries to a central conduit in the basement of the Administration Building, whence it divides and passes through sluice-gates into either one or both of the two basins of Baldwin Reservoir.

Water is delivered to these basins by means of multiple-weir flumes along the north side. It then passes across the basins and is collected on the south side at the bottom by means of openings in the conduits leading to Gate-House No. 7. There, the water is distributed by two 36-in. and three 48-in. mains into the low-service district of the city. Two other 48-in. mains leading from Gate-House No. 7, act as the suction mains for the first and second high-service pumps in Fairmount Pumping Station. The first high-service district is supplied from this station through two 30-in. mains and one 24-in. main; and the second high-service district, through one 30-in. main and one 48-in. main.

HYDRAULIC DATA

In the raw-water mains under full plant capacity, the computed velocity in the 48-in. mains is 3.3 ft. per sec., and in the 60-in. mains, 4.2 ft. per sec. With normal operating levels in the coagulation basins, the computed velocity in the channel leading from the hydraulic-jump mixing flumes to Gate-House No. 1, varies from 0.76 ft. to 1.20 ft. per sec., except at the under and over-baffles where a velocity of 2.53 ft. per sec. is reached. In the influent conduit of the coagulation basins, the computed velocity of the water is 2.17 ft. per sec. and in the effluent conduit, 2.42 ft. per sec., the latter being also approximately the maximum velocity through any of the sluice-gates. The flow through the coagulation basins is at a computed rate of 2.36 ft. per min. The computed velocity through the settled-water conduits is 2.37 ft. per sec., and through the settled-water piping leading to the filters, 2.04 ft. per sec.

It will be noted that the computed velocities of flow through conduits, sluice-gates, etc., after the water has passed through the hydraulic jumps, is less than 2½ ft. per sec. This is the value above which it was not desired to go on account of the possibility of breaking up the floc; but this velocity is exceeded if the coagulation basins are by-passed.

The computed loss of head through the filter plant, together with the actual losses obtained as the result of a series of observations made when the plant was operating at the nominal rated capacity of 165 000 000 gal. per day with

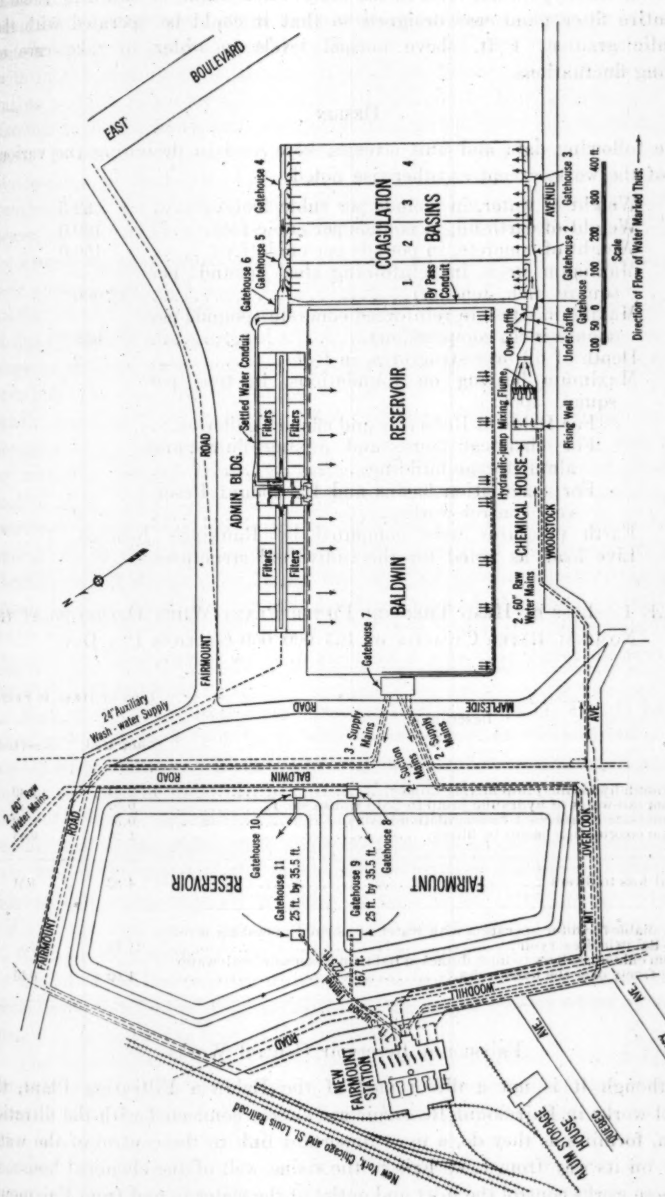

EAST
BOULEVARD
FAIRMOUNT
ROAD
N
ADMIN. BLDG.
Settled Water Conduit
Filters
Filters
Filters
Filters
BALDWIN
RESERVOIR
Gatehouse 4
Gatehouse 5
Gatehouse 6
COAGULATION
BASINS
1
2
3
4
By Pass Conduit
Over-baffle
Under-baffle
Gatehouse
Gatehouse 2
Gatehouse 3
AVENUE
Hydraulic-jump Mixing Flume
CHEMICAL HOUSE
Rising Well
2 - 60" Raw Water Mains
WOODSTOCK
100 50 0 100 200 300 400 ft.
Scale
Direction of Flow of Water Marked Thus
Gatehouse 7
24" Auxiliary Wash - water Supply
MAPLESIDE
ROAD
3 - Supply Mains
2 - Suction Mains
2 - Supply Mains
BALDWIN
ROAD
2 - 60" Raw Water Mains
Gatehouse 10
Gatehouse 8
Gatehouse 11 25 ft. by 35.5 ft.
Gatehouse 9 25 ft. by 35.5 ft.
RESERVOIR
FAIRMOUNT
MT. OVERLOOK AVE.
167 ft.
ROAD
WOODHILL
AVE.
QUINCY AVE.
NEW FAIRMOUNT STATION
ALUM STORAGE HOUSE
New York, Chicago and St. Louis Railroad

FIG. 1.—GENERAL ARRANGEMENT OF STRUCTURES AND DIRECTION OF FLOW OF WATER.

water at normal operating level in the coagulation basin, is shown in Table 1. The entire filter plant was designed so that it could be operated with the hydraulic gradient 1 ft. above normal levels in order to take care of operating fluctuations.

DESIGN

The following data and unit stresses were used in designing the various parts of the work, except as otherwise noted:

Weight of water, in pounds per cubic foot	62.5
Weight of earth-fill, in pounds per cubic foot	100.0
Weight of concrete, in pounds per cubic foot	150.0
Maximum stress in reinforcing steel (pounds per square inch, tension)	16 000
Maximum stress in reinforced concrete (pounds per square inch, compression)	650
Depth of fill over structures, in feet	2
Maximum bearing on foundations, in tons per square foot:	
For Baldwin Reservoir and effluent galleries	5
For chemical house and mixing flume and alum storage building	3
For coagulation basins and Fairmount Reservoir control works	2

Earth pressures were computed by Rankine's formula.
Live load, as noted for the individual structures.

TABLE 1.—LOSS OF HEAD THROUGH FILTER PLANT WHEN OPERATING AT THE NOMINAL RATED CAPACITY OF 165 000 000 GALLONS PER DAY.

Description.	Loss of Head, in Feet.	
	Computed.	Observed.
Loss through hydraulic-jump mixing flumes	2.04	2.40
Loss from tail-water of hydraulic jump to Gate-House No. 1	0.97	0.88
Loss from Gate-House No. 1 to coagulation basins	0.56	0.30
Loss from coagulation basins to filters	1.25	0.93
Total loss to filters	4.82	4.51
Head available for filter operation with water at normal operating level in the Baldwin Reservoir	11.75	....
Loss from effluent gallery to inlet flumes in Baldwin Reservoir, with water falling freely over the fixed weirs	1.00	0.81

Fairmount Reservoir Control Works

Although it is not a direct part of the Baldwin Filtration Plant, the control works in Fairmount Reservoir are closely connected with the filtration system, forming as they do, a most important link in the control of the water supply on its way from Lake Erie to the rising well of the chemical house.

These works control the inlet and outlet of the water to and from Fairmount Reservoir (see Fig. 1). As may be noted, each of the 60-in. raw-water mains leads to a single inlet gate-house (No. 8 or No. 10) and each gate-house, in turn, is connected to one of the two basins comprising the reservoir. These two gate-houses are cross-connected by means of a 60-in. pipe line. In conjunction with this cross-connection, the operation of the proper valves on the raw-water mains permits either of these mains to feed one or both gate-houses. Flow through each gate-house into the reservoir is controlled by a 72 by 84-in. sluice-gate. In order to prevent the water from overflowing into the street, if the sluice-gates in both gate-houses are improperly closed, each gate-house is provided with an overflow, which is directly connected to the Baldwin Road sewer, and is capable of taking the flow of one-half of the filter plant capacity.

From each gate-house, a 72-in., riveted steel pipe passes through the reservoir embankment and down the slope to the floor of the reservoir. In the section through the embankment the pipe is entirely surrounded by concrete. A cut-off wall is also provided which is connected to a concrete apron on the slope to prevent water working its way to the outside of the reservoir. The remainder of the pipe on the slope and reservoir floor rests in a concrete cradle to which it is anchored. Due to the possibility of damage by ice to the part of the pipe on the slope, provision is made near the top and bottom, by means of flanged joints, for replacing this part of the inlet pipe. The section of the pipe laying on the reservoir floor slopes upward somewhat and increases in size from 72 to 96 in. in diameter. It is laid so as to point approximately to the diagonally opposite corner of the reservoir. These three points of design—the larger end, slowing up the velocity; the upward trend, tending to widen vertical distribution; and the angle, directing the water toward the opposite corner from that in which the outlet gate-house is located—are incorporated so as to assist in a somewhat better distribution of water in the basins.

Each of the outlet gate-house structures (Nos. 9 and 11, Fig. 1) has three water passageways, 4 ft. 8 in. in width, connecting the reservoir to a central well 12 ft. in diameter. Each passageway is controlled by a 42 by 72-in. sluice-gate at the central well. To permit water to be drawn from the reservoir at different levels, a separate port, opening into each passageway, was left in the outside wall of the gate-house. One of these was at the bottom of the reservoir, one at the top, and the third midway between the top and the bottom. Screens and stop-planks are provided in each of the three passageways. The walls of the well and passageways support an operating floor approximately 5½ ft. above high water in the reservoir. The bottom of the well is connected, by means of an inverted truncated cone, to a shaft 7 ft. in diameter. This latter leads to a tunnel 7 ft. in diameter, the center line of which is approximately 28 ft. below the floor of the reservoir.

The foundation of each gate-house has approximately 6 ft. 6 in. of rubble concrete topped with 2 ft. of plain concrete. The bottom of this foundation rests on hard shale and the top is just a little above the reservoir floor. The three passageways and central well are formed by heavily reinforced concrete sections. As a protection against ice in the reservoir, all exterior concrete is faced with granite to a point approximately 5 ft. below the overflow.

The tunnels (Fig. 1) leading from the outlet gate-house shafts converge and form one tunnel of the same diameter at a wye-intersection. Continuing from this wye, the tunnel extends a distance of approximately 129 ft., where it joins a 7-ft. circular conduit, leading to the header on the suction of the low-lift pumps in Fairmount Pumping Station. The junction between each of the two vertical shafts and the tunnels is made with long-sweep radii in order to reduce loss of head.

The tunnel is bored through shale. Its cross-section is, in general, circular with a minimum thickness of shell of 12 in. At the approach to the wye, the section is made elliptical, 5 ft. 6 in. by 9 ft., with the long diameter vertical, in order to reduce to a minimum the width of unsupported roof at the wye proper. Except at the wye, where the roof becomes flat, the tunnel is composed of plain concrete. At the wye, ⅝-in. square steel reinforcing hoops were placed on 4-in. centers. Provisions were also made in the shafts and tunnel for bulkheading in case it were necessary to use compressed air.

Chemical House and Mixing Flumes

General Description.—The arrangement of the chemical house, mixing flumes, and channel is shown in Fig. 2. With the exception of the upper story of the chemical house, all the structures are of reinforced concrete of the beam-and-slab type with independent column footings. The upper story consists of brick bearing walls with steel roof trusses. The remainder of the building proper is of the skeleton type with brick curtain-walls. Basic data concerning this unit (see Fig. 2) may be listed, as follows:

Nominal capacity of the plant, in million gallons daily	165
Elevation of water surface in the rising well	245.74
Concerning a point in the channel just before the hydraulic jump:	
Depth of water, in feet	0.47
Width of three flumes, in feet (each flume)	14.56
Velocity of water, in feet per second	12.5
Elevation of water surface	242.82
Concerning a point at the foot of the slope in the mixing chambers; nominal operating conditions:	
Depth of water, in feet	3.19
Width of water passages, in feet (three each at 22 ft.)	66
Velocity of water, in feet per second	1.13
Elevation of water surface	243.69
Loss of head, in feet, between the rising well and the foot of slope in the mixing chamber	2.05
Loss of head, in feet, between the foot of slope in the mixing chambers and the coagulation basins	1.69
Distance, in feet, down stream from the top of slope in the mixing chambers, to the location of the hydraulic jump	8
Flow-line elevation in the coagulation basins	242.00
Alum Storage Bins:	
Gross capacity, in tons	450
Net capacity, in tons	350
Dissolving Tanks:	
Volume, in cubic feet per tank, to the top of overflow plank weir	157
Charge of alum required, in pounds per tank	4 000
Solution Tanks:	
Capacity of each tank, in gallons	9 600
Alum, in 5% solution, required for each tank (tons)	2
Depth, in feet, of the solution in the tanks (for 9 600 gal.)	4.62

Summary of Alum Required:	
Maximum volume of water treated, in million gallons daily	165
Alum, in grains per gallon	1
Rate, in tons per day, in which alum is applied	11.8
Speed of bucket hoist, in feet per minute (capacity, 3 000 lb.)	20
Speed of passenger elevator, in feet per minute (capacity, 2 500 lb.)	150
Alum storage capacity, in tons, required to treat 165 000 000 gal. daily with 1 grain per gallon:	
Baldwin Chemical House (30-day supply)	350
Alum Storage Building (85-day supply)	1 000
Baldwin Chemical House, on the dissolving-tank floor (20-day supply)	225
Total storage capacity (135-day supply), approximately	1 575
Height, in feet, to which bags of chemicals may be stored	3

Chemical Bin Structures.—The chemical bins (Fig. 2) occupy the major portion of the structure to the west of the rising well. They consist of two separate bins with a passageway between them, and with the top of the bins situated at the general ground level. This total net capacity is about 350 tons. The westerly bin extends the full width of the building, and the easterly one, a little less than one-half this distance. They are continuous, with seven hoppers in the large bin and three in the small one. The minimum side slope is approximately 45 degrees. The bins discharge through 12 by 14-in. steel chutes equipped with single undercut gates, operated with a chain-and-link mechanism. These chutes discharge into a bucket on a small industrial or transfer car that is spotted in a common passageway beneath the bins. The cover over the alum bins forms the floor of the receiving room (see Section *D-D,* Fig. 2). In this floor there is one manhole to approximately 90 sq. ft. of horizontal bin surface.

Chemical Tanks and Piping.—On the upper floor of the chemical house are located the tops of six dissolving and four solution tanks. These are divided evenly into two independent groups, each of which is connected with a double orifice box unit. The arrangement is such that the central dissolving tank in each group is capable of feeding either solution tank. A system of piping conveys the solution from the solution tanks to the orifice boxes and from there to the lead-lined distributing trough immediately over the rising well. Piping is also provided to convey the solution to a point of secondary application in Gate-House No. 6 which is in the outlet conduit from the coagulation basins.

All chemical piping is flanged cast iron and all valves are of acid bronze. In all cases in which this piping is joined to chemical tanks, special wall castings, no parts of which come in contact with the chemical solution, were provided. This will permit complete replacement of the chemical lines at any future time and at a minimum expenditure.

Hydraulic-Jump Section.—The raw water is introduced to the mixing-flume section, at its westerly end (see Fig. 2), by means of a rising well which extends entirely across the width of the chemical house. The raw water enters the rising well from two 60-in. mains.

Flowing from the upper pool, the water passes through three hydraulic-jump mixing flumes. These flumes have expanding sides and sloping bottoms, and are preceded by horizontal throats of constant width with curved entrances.

At the foot of the slopes, they lead into a converging horizontal flume (see Fig. 2). Provisions have been made in each throat for a double row of stop-planks in order to cut off the flow of water through the flume, if desired. The flumes are housed within a one-story building; walk-ways, conveniently located, permit an adequate view of the hydraulic jumps.

Just beyond the weir planks for controlling the tail-water depth, the converging shallow flume is transformed to a conduit section (see Fig. 2). This is accomplished by dropping the floor about 9 ft. and narrowing the sides, thus producing a conduit section about 19 ft. wide and 13 ft. deep leading to Gate-House No. 1. This conduit section, which is about 90 ft. long, has a pair of under and over-baffles near each end.

In order to present a more pleasing external appearance, the exterior walls of the chemical house were extended through to Gate-House No. 1, and the area between these walls and the conduit was roofed over at the level of the top of the conduit. Over this entire area an earth-fill, 2 ft. deep, was provided.

Parts of the space between the conduit and the exterior walls were utilized for garage, tool-room, and covered passageway. This passageway is part of a complete system, leading from the chemical house, along the coagulation basins, to the filter building. Beneath it is a tunnel in which are located the steam and water lines, secondary chemical feed pipes, and conduits for light, power, and telephone cables.

Basis of Design.—Except in the case of water passages, where maximum water heights are used, the usual live load is 75 lb. per sq. ft. On the floor where the dissolving tanks are located—owing to the possibility of storing sack alum—a live load of 250 lb. per sq. ft. is used. The floor over the alum storage bins is designed to carry a motor truck which, when loaded, weighs about 7 tons.

Expansion joints, extending down through the foundation, are provided at six places in the chemical house and mixing flumes. They are placed where pronounced changes are made in the general type of the structure, and, in addition, at one or two other places, so that in no case is there a distance between joints greater than 82 ft. The shape and proportions of the hydraulic-jump flumes, as well as the dissolving and solution[5] tanks, are based on experiments conducted by the Engineers of the Cleveland Water Department.[6]

Coagulation Basins

The coagulation basins shown in Fig. 1, extend for a distance of nearly 500 ft. from the east wall of Baldwin Reservoir. Data pertaining to these basins are, as follows:

Over-all area, in square feet, including conduits and gate-houses (457 ft. by 747 ft.)..	341 379

---

[5] "Rate of Solution of Sulphate of Alumina," by J. W. Ellms, A. G. Levy, and L. A. Marshall, *Journal*, Am. Water Works Assoc., July, 1921.

[6] "The Hydraulic Jump as a Mixing Device," by A. G. Levy and J. W. Ellms, *Journal*, Am. Water Works Assoc., January, 1927.

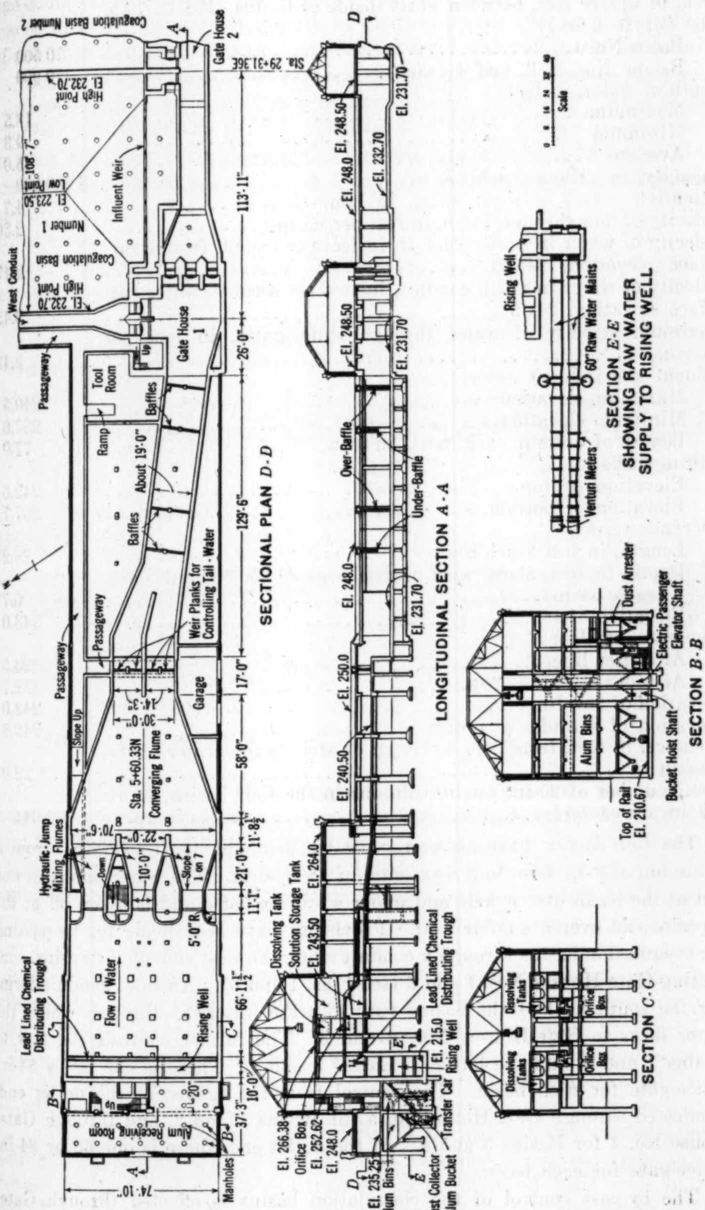

SECTIONAL PLAN D-D
Coagulation Basin Number 2
Coagulation Basin Number 1
West Conduit
High Point El. 232.70
Low Point El. 223.50
Influent Weir
108'-7"
Gate House 2
Gate House 1
Sta. 29+31.36E
Passageway
Tool Room
Ramp
Baffles
About 19'-0"
Weir Planks for Controlling Tail-Water
Garage
Slope Up
Sta. 5+60.33N
Converging Flume
Hydraulic-Jump Mixing Flumes
Down
Slope 1 on 7
5'-0" R.
Lead Lined Chemical Distributing Trough
Flow of Water
Rising Well
Alum Receiving Room
Up
Manholes
74'-10"
37'-3"
10'-0"
66'-11½"
11'-1"
21'-0"
11'-8½"
58'-0"
17'-0"
129'-6"
26'-0"
113'-11"
14'-3"
30'-0"
22'-0"
70'-6"
10'-0"
20"
A
B
C
D
E
LONGITUDINAL SECTION A-A
El. 266.38
Orifice Box
El. 252.62
El. 248.0
El. 235.25
Alum Bins
Dust Collector
Alum Bucket
Transfer Car
El. 215.0
Rising Well
Lead Lined Chemical Distributing Trough
Dissolving Tank
Solution Storage Tank
El. 243.50
El. 264.0
El. 240.50
El. 250.0
El. 248.0
El. 231.70
Under-Baffle
Over-Baffle
El. 248.50
El. 231.70
El. 232.70
El. 248.0
El. 248.50
El. 231.70
SECTION C-C
Dissolving Tanks
Orifice Box
SECTION B-B
Top of Rail El. 210.67
Bucket Hoist Shaft
Alum Bins
Electric Passenger Elevator Shaft
Dust Arrester
SECTION E-E SHOWING RAW WATER SUPPLY TO RISING WELL
Venturi Meters
60" Raw Water Mains
Rising Well
Scale
0 8 16 24 32 40

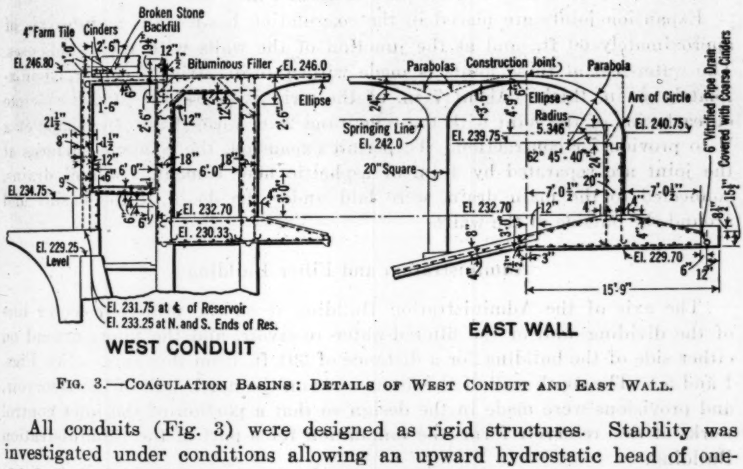
FIG. 2.—CHEMICAL HOUSE: GENERAL PLAN AND SECTIONS.

Area, in square feet, between weirs inside of basins (108 ft. 7 in. by 661 ft. 6 in.):	
Basin No. 1	70 500
Basins Nos. 2, 3, and 4 (each)	70 900
Depth of water, in feet:	
Maximum	18.5
Minimum	9.3
Average	15.0
Capacity, in gallons, each basin	8 200 000
Detention period between weirs, in hours	4.7
Velocity of flow through basin, in feet per minute	2.36
Velocity of water in south conduit, in feet per second (water surface elevation, 242.5)	2.17
Velocity of water in north conduit, in feet per second (water surface elevation, 241.5)	2.42
Maximum velocity of water through sluice-gates, in feet per second	2.43
Effluent and influent weirs:	
Maximum elevation	240.5
Minimum elevation	237.0
Length of weirs in each basin, in feet	77.0
Stilling-baffle:	
Elevation of top	242.5
Elevation of bottom	237.71
Overflow weirs:	
Length, in feet (each basin)	28.27
Depth, in feet, above weir to discharge 41 250 000 gal. daily (each basin)	0.77
Elevation of top	243.0
Elevation of floor:	
At center line	223.5
At sides	232.7
Elevation of flow line	242.0
Elevation of intrados at crown	245.5
Distance, in feet, from floor to crown of arch at the center of the basin	22.0
Total number of 20-in. square columns in the four basins, 15 ft. 9 in. on centers	1 041

The four basins have an average water depth of 15 ft., varying from a minimum of 9 ft. 4 in. to a maximum of 18 ft. 6 in. The water enters at one end of the basin over a weir and under a stilling-baffle and is taken off at the opposite end over a similar weir. Provisions have been made for by-passing the coagulation basins through a conduit along the west side of the basins, connecting Gate-Houses Nos. 1 and 6 (see Figs. 1 and 3). An inlet conduit forming the south wall of the basins furnishes the waterway through which the water flows to Gate-Houses Nos. 2 and 3. The former controls the flow to Basins 1 and 2, and the latter to Basins 3 and 4. There is one 60 by 84-in. sluice-gate for each basin. The control from the basins, at the outlet end, is effected through Gate-House No. 5 for Basins 1 and 2, and through Gate-House No. 4 for Basins 3 and 4. At the outlet end, there is one 60 by 84-in. sluice-gate for each basin.

The by-pass control of the coagulation basins is effected through Gate-Houses Nos. 1 and 6, where the water may be shut off entirely from the north and south conduits. When the by-pass is in use all the water flows through two 60 by 84-in. sluice-gates in each of the gate-houses. Under the condition of maximum flow, a velocity through these gates of about 3¾ ft. per sec. results.

Each basin is of reinforced concrete construction with groined arch roof, reinforced sloping floor, and independent column footings. (See Fig. 4.) The floor consists of two parts, a sub-floor 4 in. thick and a finished floor 8 in. thick, the latter being reinforced with ½-in. square deformed bars on 15-in. centers in both directions. The sub-floor butts against the walls and column bases. The finished floor rests on a ledge at the walls and at the column footings, and all floor joints are filled with asphalt to prevent leakage. The sloping floor performs the dual function of facilitating cleaning and of reducing the height of the wall sections.


Broken Stone
Backfill
4"Farm Tile
Cinders
El. 246.80
1'-0"
2'-6"
19½"
12"
½" Bituminous Filler
El. 246.0
Ellipse
1'-6"
2'-6"
12"
2'-6"
21½"
8"
1½"
12"
6"
6' 0"
18"
6'-0"
18"
9"
El. 234.75
El. 232.70
3'-0"
4"
6"
El. 230.33
El. 229.25
Level
El. 231.75 at ℄ of Reservoir
El. 233.25 at N. and S. Ends of Res.
Parabolas
Construction Joint
Parabola
24"
3'-6"
4'-9"
6"
Ellipse
Arc of Circle
Radius
5.346'
El. 240.75
Springing Line
El. 242.0
El. 239.75
62° 45'-40"
24"
20" Square
20"
7'-0½"
7'-0½"
El. 232.70
12"
4"
4"
3'-0"
8"
7½"
3"
El. 229.70
6"
12"
8½"
15½"
15'-9"
6" Vitrified Pipe Drain Covered with Coarse Cinders

WEST CONDUIT

EAST WALL

FIG. 3.—COAGULATION BASINS: DETAILS OF WEST CONDUIT AND EAST WALL.

All conduits (Fig. 3) were designed as rigid structures. Stability was investigated under conditions allowing an upward hydrostatic head of one-third the height of the water in the basin. In the case of the north conduit, recognition was taken of the possible removal of the earth backing at some future time to take care of adjacent construction, and the conduit was designed accordingly. At the by-pass conduit, the usual barrel arch was designed as a cantilever, extending from the roof of the conduit. Because of the frequency with which it has been observed that arch action does not always take place between the barrel and groined arches, all dividing walls between the basins were designed on the basis of an unbalanced arch thrust. The groined arches are tied together in groups of four and reinforced with two ½-in. square bars on either side of the crown. All manholes are tied securely to the groined arch roof with steel reinforcement extending well into the arches. This is required particularly in the case of the manholes supporting the mechanism for the mud-valves, in order to prevent frost action between the roof and manholes lifting the mud-valves off their seats in the floor of the basin, and thus permitting water to leak into the drains. The columns supporting the roof are of five different lengths, varying from 10 ft. 8 in. to 17 ft. 11 in. In the 10 ft. 8-in. and 12 ft. 8-in. columns, no reinforcement is used. The remaining columns are reinforced with four ¾-in. square bars, with ¼-in. round ties spaced 12 in. center to center. Six stiffening walls are placed in each basin, parallel to the long axis, and near the center of the basin.

Each coagulation basin is provided with six mud-valves for cleaning and draining. These valves permit flow into a lateral drain placed immediately under the floor of the basin and along the longitudinal axis. Each drain is connected into the main drain header near the north end of the basin.

To assist in cleaning, 2½-in. hose gates spaced about 50 ft. from center to center are provided on each side of each basin. They are connected to 6-in. water mains placed on the long walls of the basins.

Expansion joints are placed in the coagulation basin walls at intervals of approximately 60 ft., and at the junction of the walls with the gate-houses. The water-seal at these joints is made with a 16-in. strip of copper, approximately $\frac{1}{16}$ in. thick. About 7 in. of the strip is embedded in the concrete on each side of the joint with the remaining 2 in. formed into the shape of a **V** to provide for contraction. To permit expansion, the concrete surfaces at the joint are separated by ⅜ in. of asphaltic fiber cement. Subsoil drains, connected to the main drain, were laid under the floors of the basins and around the outside of the walls.

Administration and Filter Building

The axis of the Administration Building is on the projected center line of the dividing wall of the filtered-water reservoir, and the filters extend on either side of the building for a distance of 391 ft. from this axis. (See Figs. 1 and 5.) The south wall of the filters rests on the north wall of the reservoir, and provisions were made in the design so that a portion of the inlet control works of this reservoir forms the foundation for a part of the Administration Building.

Administration Building.—The Administration Building is a skeleton steel structure 166 by 71 ft. in plan and, approximately, 51 ft. in height from the main floor to the under side of the roof trusses. Below the main floor (Elevation 244) are a basement and an intermediate floor. The general offices are on the main floor, with the store rooms, transformer room, pipe shop, blacksmith shop, and chlorine room on the floor below (Elevation 234). The main piping, carrying the raw and filtered water, is on the lowest or basement floor. The floor containing the laboratories, general storeroom, and machine shop, and the drip floor, located just below the wash-water tanks, are above the main floor. Access is furnished to all these floors by stairs and a freight elevator.

The architectural arrangement is such that an elaborate stone double stairway leads to the main entrance to the building (Fig. 5), and advantage was taken of the space under this stairway for locating the wash-water pump-room. This room is 21 ft. 6 in. by 51 ft. 6 in. in plan, and access to it is by means of a tunnel from the basement of the Administration Building.

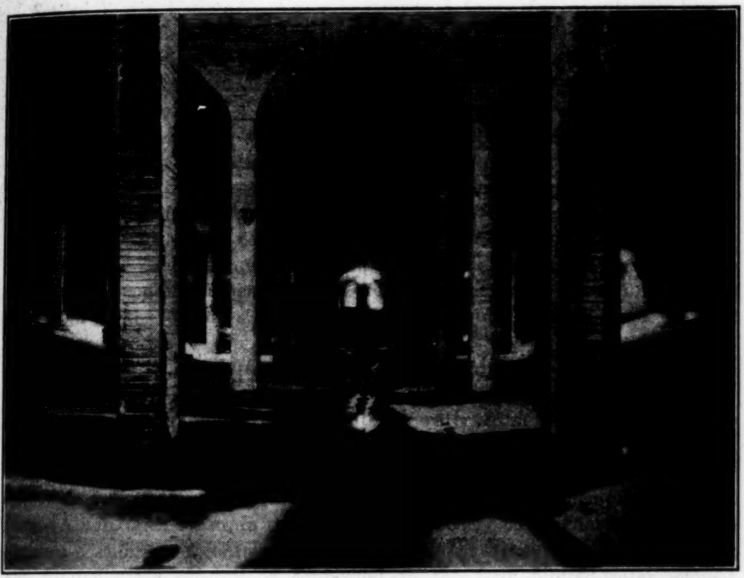


FIG. 4.—INTERIOR VIEW OF ONE OF THE COAGULATION BASINS, BALDWIN FILTRATION PLANT.

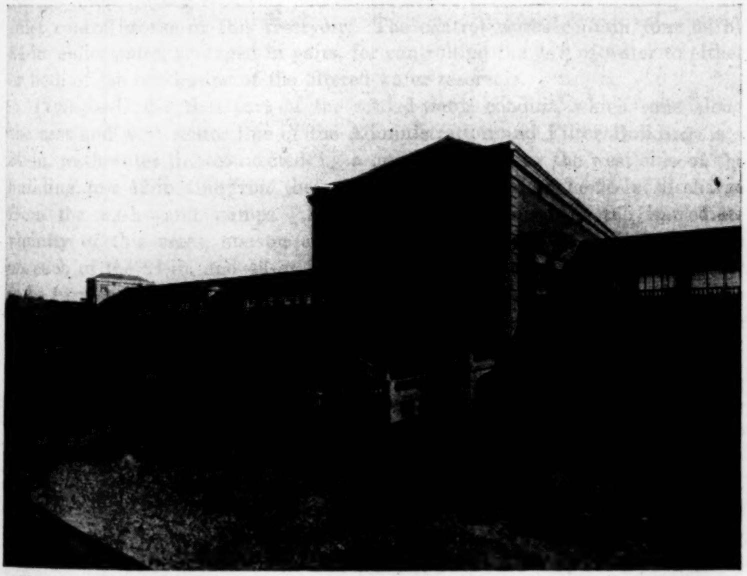


FIG. 5.—VIEW OF MAIN ENTRANCE TO ADMINISTRATION AND FILTER BUILDING, BALDWIN FILTRATION PLANT.

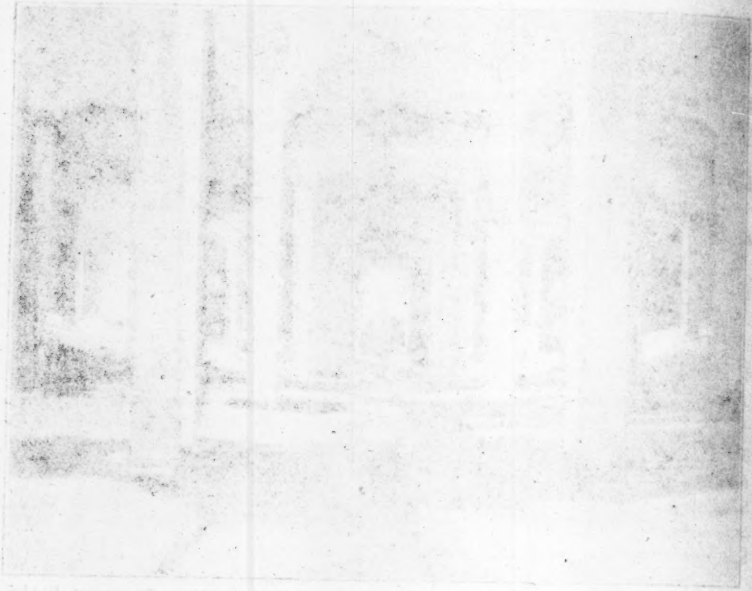


Fig. 4.—[illegible]

Fig. 5.—View of Main [illegible] Buildings [illegible]

Settled Water Conduit.—Water is conveyed from the coagulation basins through a concrete conduit, 8 ft. wide by 13 ft. 5 in. high, north of and adjacent to the east wing of the filter building. (See Fig. 1.) This conduit terminates in a section, 14 ft. 5 in. wide by 13 ft. 5 in. high, just within the Administration Building. The top of this section forms a part of the floor at Elevation 234.0. This latter conduit is connected to the conduit that runs the length of the pipe galleries, by a short section of double-barreled conduit beneath the floor at Elevation 234.0. Within the pipe gallery, the conduit is 7 ft. wide by 8 ft. high. In the Administration Building, it is 11 ft. 3 in. wide by 5 ft. high. The conduits within the pipe galleries are connected to that within the Administration Building by means of transition sections. The change in the conduit dimensions was brought about by the need of conserving head-room in the Administration Building and width of gallery in the pipe galleries themselves.

Filtered Water Conduit.—Along the north and south center line of the Administration Building and resting upon the basement floor, is a reinforced concrete conduit for conveying the filtered water from the effluent galleries to the filtered-water reservoir. From the end of the conduit nearest the settled water conduit, a 54-in. by-pass is connected to the latter. This by-pass is controlled by two 48-in. valves, which are arranged with a spacer piece between them. A 2-in. open drain, in the bottom of the spacer piece, guards against the slightest possibility of contamination of the filtered water by the settled water. The filtered-water conduit is connected to the four filtered-water effluent galleries under the filters by 54-in. cast-iron pipe controlled by 48-in. valves. At the filtered-water reservoir end of the conduit, connection is made to the inlet control works of this reservoir. The control works contain four 60 by 84-in. sluice-gates, arranged in pairs, for controlling the flow of water to either or both of the two basins of the filtered-water reservoir.

Piping.—Under that part of the settled-water conduit, which runs along the east and west center line of the Administration and Filter Building, is a 30-in. wash-water line connected, by a cross located near the west side of the building, to a 42-in. line from the wash-water tanks and to the 24-in. discharge from the wash-water pumps. Four valves are grouped in the immediate vicinity of this cross, one on either side of it on the 30-in. line and one on each of the 24-in. and 42-in. lines. This arrangement permits wash-water to be furnished either from the tanks or from the pumps to the filters in either wing of the filter building. A man-hole and drain are also provided in the 30-in. wash-water line. In the 24-in. pump discharge, a Venturi meter is provided for measuring the flow of wash-water from the pumps.

A 30-in. cast-iron suction pipe is encased in concrete, under the floor of the basement. It continues under the floor of the tunnel and connects to the wash-water pump-room. This suction pipe is connected to sumps in the floors of two of the effluent galleries, each branch being provided with a 30-in. valve.

Intermediate Floor.—The intermediate floor at Elevation 234.0, is located at the level of the top of the settled-water conduit. On this floor are rooms for storing and applying chlorine, a transformer room, and pipe and blacksmith shops. All deliveries to the Administration and Filter Buildings are made to this floor at a doorway beneath the main entrance. This door is reached by a semi-circular roadway passing under this entrance.

Main Floor.—On the main floor at Elevation 244.0, is the operators' room, in which are located various gauges. Adjoining this room is a dining room and kitchen for the use of employees. A drafting room, locker room, toilets, and space for a freight elevator complete the west half of this floor. Across from the operators' room in the southeast corner of the main floor, is an observation room, in which is located, open to public view, one of the system of weirs which admit water to Baldwin Reservoir. Adjacent to this room is a small assembly room. In the northeast corner of the building is a group of offices occupied by the superintendent and clerical forces.

Laboratory Floor.—On the laboratory floor at Elevation 258.67, are located the laboratories, machine shop, stock room, toilets, and locker room. The rooms used for chemical and bacteriological work include two chemical laboratories for water analysis, one for coal and oil testing, a preparation room for media, a bacteriological laboratory, a microscopic room, an incubator room, a refrigerator room, a photographic dark room, a library, and chemical storage rooms.

Wash-Water Tanks.—The two wash-water tanks are located on a steel grillage in the upper part of the Administration Building, with the bottom of the tanks at Elevation 281.12, or about 44 ft. above the maximum height of the wash-water troughs in the filters. These tanks are of steel-plate construction, each 60 ft. in diameter and 12 ft. deep, and are intended to operate with a maximum of 10 ft. of water. At this depth the capacity of the two tanks is 423 000 gal., or approximately the quantity required for six filter washes. A 36-in. cast-iron pipe leads from the bottom of each tank into a header, which discharges into a 42-in., vertical, pipe line, leading to the basement where it joins the 30-in. wash-water line in the pipe galleries. The pipe from each tank is provided with a 36-in. valve for controlling the flow. A 16-in. cast-iron overflow is installed in each tank. The entrance to this overflow is over a bellmouth piece with its weir edge placed 10.25 ft. above the bottom of the tank. In order to reduce the tendency to suck air caused by vortex action under low-water conditions, an inverted steel cone 3 ft. 10 in. in diameter and with the sides forming an angle of 45° to the base, was placed at the entrance to each 36-in. outlet. The apex of the cone is set in the plane of the bottom of the tank. In order to facilitate the maintenance of the steel tanks and the grillage, as well as to take care of condensation from the tanks, pipe, etc., a reinforced concrete floor with a proper system of drainage is placed under the tanks at an elevation of approximately 272.0 ft.

Filter Building.—The filters are housed in two wings of the Administration and Filter Building. (See Fig. 5.) These wings are each approximately 360 ft. long by 130 ft. wide and extend at right angles to the major axis of the Administration Building. Statistics relating to these units are as follows:

Nominal capacity of filter beds, in million gallons per day......	165
Number of filter beds	40
Capacity, in million gallons per day per filter	4.16
Rate of filtration, in million gallons per day per acre	125

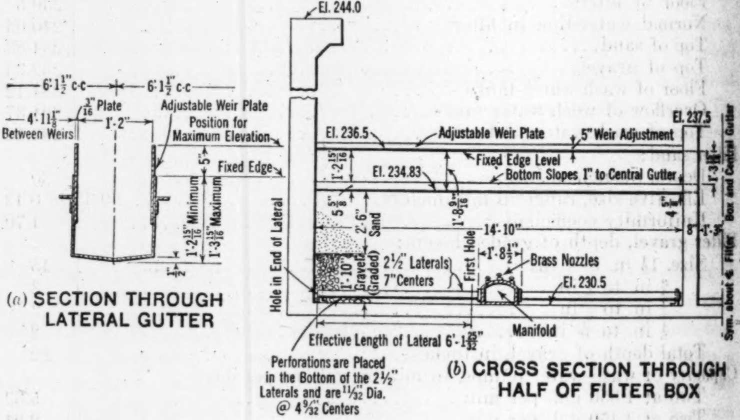
Clear distance between weirs of the wash-water troughs, in feet	4.93
Capacity of wash-water troughs, in inches of rise per minute:	
With adjustable weir plates down	24
With adjustable weir plates up	33
Probable practical rates of rise, in inches of rise per minute:	
With adjustable weir plates down	21
With adjustable weir plates up	26
Wash-water capacity of main gutter, in inches of rise per minute.	33
Maximum velocity of flow through settled-water conduits, in feet per second	2.42
Maximum velocity of flow through 24-in. settled-water pipes, in feet per second	2.04
Elevations:	
Edge of wash-water troughs:	
Maximum	236.92
Fixed	236.50
Top of central gutter	237.50
Bottom of central gutter (sloped from Elevation 230.75)	230.50
Floor of effluent galleries	216.0
Floor of filters	230.5
Normal water-line in filters	240.63
Top of sand	234.83
Top of gravel	232.33
Floor of wash-water tanks	281.12
Overflow of wash-water tanks	291.37
Top of wash-water tanks	293.12
Filter sand:	
Depth, in inches	30
Effective size, range in millimeters	0.35 to 0.42
Uniformity coefficient	1.70
Filter gravel, depth of graded layers:	
Size, 1½ in. to ¾ in	15
¾ in. to ¼ in	3
¼ in. to ⅛ in	2
⅛ in. to 1/16 in	2
Total depth of gravel, in inches	22
Capacity of wash-water pumps, in million gallons per day:	
Two at 1 850 gal. per min	5.33
Two at 3 450 gal. per min	9.94
Total capacity of four pumps	15.27
Capacity of two wash-water tanks (water, 10 ft. deep) each tank, in gallons	211 500
Diameter of wash-water tanks, in feet	60
Total weight of one full tank (10 ft. of water), in tons	946
Average quantity of water required to wash one filter, in gallons.	71 000

Each group of ten filters is built upon the roof of an effluent gallery or small filtered-water reservoir into which these filters discharge. The filter operating floor is located at the level of the top of the filters and spans the pipe gallery.

Filter Tanks—The filter tanks were designed to be built as monoliths. Each unit has a sand area of 1 450 sq. ft. divided into two equal areas by a center gutter. The outside walls of the tanks are designed as vertical slabs held at the bottom by the tank floor and at the top by the filter walks, acting as horizontal beams and as ties. The walks extend along the top of the four outside walls, over the central gutter, and at right angles to the center walk.

Wash-water gutters, running at right angles to the center gutter, are designed and arranged as shown in Fig. 6. Longitudinally, the slope of the bottom is 1 in. in the full length of the gutter.

The strainer system of the filter tanks is of the perforated-pipe manifold type, with the manifold located under the central axis of the sand bed in each half of the filter. The manifolds for each half filter consist of four independent castings, each having a central source of water supply. Each of the manifold castings supplies forty-two cast-iron laterals, arranged as shown in Fig. 6. In order to wash the otherwise dead area over the manifold, $\frac{3}{8}$-in. brass nozzles were provided, one nozzle being furnished for approximately 52 sq. in. of sand area over the manifold. The total area of the holes in the strainer system is 0.309% of the area of the sand bed. The area of the holes in one lateral is 36% of the cross-sectional area of the lateral, and the area of the laterals fed by one manifold is 71.4% of the cross-sectional area of the manifold. The effective length of the lateral is 29.5 times its diameter.


6'-1½" c-c
6'-1½" c-c
3/16" Plate
4'-11⅛" Between Weirs
1'-2"
Adjustable Weir Plate
Position for Maximum Elevation
5"
Fixed Edge
1'-2 15/16" Minimum
1'-3 15/16" Maximum
½
El. 244.0
El. 236.5
Adjustable Weir Plate
5" Weir Adjustment
El. 237.5
Fixed Edge Level
El. 234.83
Bottom Slopes 1" to Central Gutter
1'-2 15/16
1'-3 1/16
5⅝"
2'-6" Sand
1'-8 9/16"
4½"
14'-10"
8"
1'-3"
Hole in End of Lateral
1'-10" Gravel (Graded)
2½" Laterals 7" Centers
First Hole
1'-8½"
Brass Nozzles
El. 230.5
Manifold
Effective Length of Lateral 6'-1 25/32"
Perforations are Placed in the Bottom of the 2½" Laterals and are 11/32" Dia. @ 4 9/32" Centers
Sym. about ℄ Filter Box and Central Gutter

(a) SECTION THROUGH LATERAL GUTTER

(b) CROSS SECTION THROUGH HALF OF FILTER BOX

FIG. 6.—GENERAL CROSS-SECTION OF ONE-HALF OF A COMPLETED FILTER TANK.

The strainer system was designed in accordance with the results of experiments by one of the writers,[7] which were an extension of those made by Harry N. Jenks,[8] M. Am. Soc. C. E. The original design contemplated the use of flange connections for the laterals, but, on the basis of bids submitted, it was found that a considerable saving could be made by the use of the bell-and-spigot joint. To eliminate the possibility of slippage at the lead joint, a collar is cast on the lateral near the closed end. Cast-iron lateral chairs were designed to be slipped over the ends of the laterals as far as the collar and then to be bolted to the filter tank bottom. The manifolds are practically uniform in cross-sectional area. They are provided with hand holes near the end of each casting and with a 12-in. manhole at the center just over the inlet for the water supply.

---

[7] "Design of Perforated Pipe Strainer System," by J. W. Ellms, M. Am. Soc. C. E., *Journal*, Am. Water Works Assoc., December, 1927.

[8] "An Investigation of the Efficiency of the Perforated Pipe Underdrain System in the Washing of Rapid Sand Filters," by Harry N. Jenks, M. Am. Soc. C. E., July, 1920.

Effluent Galleries.—The effluent galleries are four in number and each is 352 ft. 6 in. long. The two north galleries are each 47 ft. 6 in. wide and the south galleries, 45 ft. 3 in. wide. The distance from the floor to the under side of the roof is 13 ft. 3 in. The capacity of the four galleries is about 6 000 000 gal. Each gallery is provided with two 12-in. drains. These drains empty into the main drain and each is provided with two valves in series. A 24-in. flanged access manhole was provided in the common wall between the effluent and pipe galleries at about the mid-point in the length of the wall. At the ends of the galleries next to the Administration Building, sight wells, 4 by 5 ft. in plan, open into the effluent galleries. They permit observation of the filtered water during operation, and provide means of taking out any of the piping in the effluent galleries, in case such a need should arise. The piping in the effluent galleries, in conjunction with the strainer system, acts as the collecting system for the filtered water, conveying it from the filters through the controller in the pipe gallery and then back again into the open effluent gallery. The same system of piping in the effluent galleries serves for the distribution of the wash water during the washing process.

The floor of each effluent gallery rests on shale and is designed for a minimum thickness of 10 in. It is reinforced on the bottom with $\frac{5}{8}$-in. square bars, spaced on 6-in. centers and running in both directions. The footings for the columns supporting the effluent gallery roof with its superimposed loads, rest directly on this floor. To transfer the loads to the columns, large beams and girders were provided, those coming under the center line of the filters being 25 in. wide and 52 in. deep. The longitudinal walls of the effluent galleries were designed as slabs tied into the floor and roof. The walls at the extreme east and west ends were similarly designed, advantage being taken of the passive resistance of the earth against the wall. At the Administration Building, due to the lack of passive earth resistance to counteract the load transmitted by the wall to the roof, and to the existence of an expansion joint 73 ft. from the wall, special reinforcement was designed for the roof to transfer the load into the longitudinal walls.

To provide for temperature changes, expansion joints were placed 71 ft. apart. The joints are located immediately below the dividing plane between each group of two filters. The expansion joints extend from north to south across the entire building and from the roof of the effluent galleries down through the foundations. Copper plates were used at all expansion joints to prevent leakage of water. These plates were carried from the bottom of the foundations to the top of the filter tanks.

Pipe Galleries.—The pipe galleries occupy that space in the filter building between the two rows of filter tanks, and extend from the level of the effluent gallery floor to the operating floor. There are two galleries, each having the same length as the effluent galleries. The pipe galleries are 27 ft. wide and approximately 28 ft. high. In these galleries are the settled-water conduit and wash-water pipe, with connecting piping for each individual filter, and the filter rate controllers. An 8-in. water-pressure line and a steam-heating line suspended from and approximately 3 ft. below the operating floor, run the entire length of the two pipe galleries and across the Administration Building.

The settled-water conduit is of reinforced concrete, rectangular in section, 7 ft. wide and 8 ft. high. It is supported by concrete columns spaced approximately 9 ft. apart. Expansion joints are placed nearly opposite those in the effluent galleries and 71 ft. apart. All expansion joints are located midway between columns, the conduit accordingly acting as a cantilever. The top of the conduit is at Elevation 237.0, and serves as a walkway along the entire length of the pipe gallery. This walkway is easily accessible from the floor level in the Administration Building at Elevation 234.0, the top of the transition section of the conduit at the entrance to each gallery serving as a ramp.

The main drain is a rectangular concrete conduit, the top of which is reinforced and serves as a part of the pipe-gallery floor. This conduit is designed to have the sides bear directly against shale. It is 5 ft. wide and, at the shallowest section, 5 ft. deep. The bottom has a slope of 0.00251 ft. per ft. When washing two filters at the same time, there is a slight upward pressure on a part of the drain roof. Because of this fact the floor drainage from the pipe gallery is not designed to discharge into the main drain, but to empty into a 12-in. cast-iron sub-drain. This sub-drain is laid with open joints, in crushed stone, under the bottom of the main drain, and discharges into it at a point on the slope beyond the filter building, where there is no danger of water backing into the pipe gallery.

The pipe-gallery floor is 10 in. thick and rests on the shale. It is of plain concrete, and slopes 3 in. in about 11 ft. from the effluent gallery walls toward the top of the main drain. A vertical drop of 3 in. is made at the junction of the pipe-gallery floor proper with the top of the main drain, thus forming a distinct drainage channel along the longitudinal center of the pipe gallery. This channel discharges into the floor drain sumps, spaced 140 ft. apart. The invert of the drainage channel slopes 3 in. in 70 ft. toward these sumps. These sumps are connected by 6-in. cast-iron pipe to the 12-in. cast-iron sub-drain, previously mentioned.

The wash-water pipe line is a 30-in. cast-iron bell-and-spigot main, located under the settled-water conduit, and supported on concrete beams spanning between the columns supporting this conduit. At the west end of the pipe gallery, the wash-water pipe line is connected to a 24-in service supply main, and this furnishes an additional or emergency source of wash-water supply.

The control of this supply of wash water is by means of an altitude valve. If this valve fails to close when the wash-water tanks are full, the supply is closed by an electrically operated gate-valve between the altitude valve and the wash-water tanks. This emergency valve may be controlled by floats in the wash-water tanks or by a push-button switch. It may also be operated manually by means of a hand wheel. Just outside the building, in a valve vault, is a 24-in. pressure-regulating valve, a 24-in. check valve, and an 8-in. emergency connection to a higher pressure water-supply main. The pressure-regulating valve is used to reduce the pressure in the service main to that required in the wash-water system. The 24-in. check valve was required to prevent the water from the 8-in. emergency connection from backing up into the lower pressure 24-in. service main.

The system of piping at each filter consists of a 24-in. influent line controlled by a valve of the same size, a 24-in. effluent line containing a filter rate controller and a 24-in. valve, a 24-in. wash-water line controlled by a valve of the same size, a 30-in. drain line with a 30-in. valve, and an 8-in. filtered-water waste line and valve. All the valves are of the double-disk, parallel-seat type, and all are hydraulically operated except the hand-operated 8-in. filtered-water waste valve.

All valves, 24 in. and larger, where placed horizontally on their sides, are provided with rollers and scrapers. The 30-in. drain valves which have their disks in a horizontal plane, are provided with special heavy bronze mounted slides, operating on bronze tracks built in the body and bonnet of the valve, in order to prevent the wearing of the gate and seat rings. Just before the disks come to a stop, the tracks slope to permit them to drop into a closing position before the wedging mechanism operates.

Special precautions were taken to prevent the scouring of the brass lining by iron rust in the hydraulic valve cylinders. The cast-iron piston, which consists of two rings with a spacer, is provided with cup leathers, and is secured to the stem with bronze lock nuts. The circumference and upper and lower faces of the spacer or piston head, not in contact with the cup leather and followers, are covered with a brass facing, forced into place and securely fastened to the spacer.

The 8-in. pressure line running longitudinally through the pipe galleries, supplies water for operating the hydraulic valves. To avoid shutting down a considerable number of filter units due to a break in this line, it is fed from either end, and valves are placed at the quarter-points. At each end of the line, pressure-regulating valves are provided to reduce the pressure to that required for operating the hydraulic valves. The waste from all the hydraulic valves of each filter is piped to the central gutter of the particular filter to which they belong.

Operating Floor.—The operating floor is at the same elevation as the top of the filter tanks. It is of reinforced concrete, 7 in. thick, with quarry tile surfacing. It is supported on columns resting on the outer walls of the settled-water conduit, and at the filters on ledges constructed as a part of the individual filter tanks. Access from this floor to the top of the settled-water conduit as well as to the floor of the pipe gallery is provided at the ends of the pipe galleries by reinforced concrete stairs.

To provide light and air in the pipe galleries, openings 7 ft. wide by 17 ft. long, are located in the operating floor. These openings are centered on the dividing line between adjacent pairs of filter tanks. An ornamental iron rail follows the outline of the openings in the floor and continues, between openings, along the edge of the filter tanks. Easy access from the operating floor to the walks over the filter tanks is provided by means of gates.

Baldwin Reservoir

General Description.—As previously noted, the Baldwin Reservoir was originally planned as a distributing reservoir for the low-service district. The original design was based on a 130 000 000-gal. open reservoir. The excavation was about three-fourths completed when it was decided by the City of Cleveland to construct a filtration plant immediately adjacent to the reservoir, using it as the clear-water basin for this plant. Due to the fact that the excavation was so far advanced, the elevation of the bottom as well as the slope of the floor and the area to be occupied by the reservoir was fixed in a general way.

Baldwin Reservoir, as constructed, is about 1 035 ft. by 550 ft. in plan, and 40 ft. from the floor to the crown of the groined arch roof. The reservoir is divided into two basins of equal size. The maximum depth of water is 36.5 ft. in each basin, and the two basins have a total capacity of 135 000 000 gal. In all, there were 1 196 columns, the dimensions of which are shown in Fig. 7.

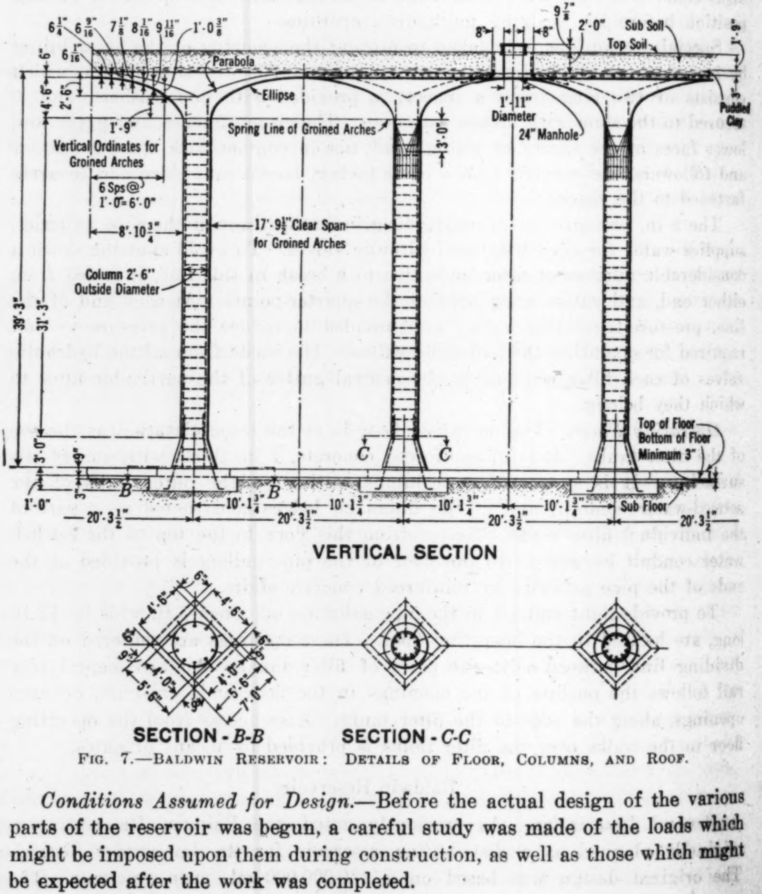

6 1/4"
6 9/16"
7 1/8"
8 1/16"
9 11/16"
1'-0 3/8"
6 1/16"
6"
4'-6"
2'-6"
Parabola
Ellipse
1'-9"
Vertical Ordinates for Groined Arches
6 Sps @ 1'-0"= 6'-0"
8'-10 3/4"
17'-9 1/2" Clear Span for Groined Arches
Spring Line of Groined Arches
3'-0"
Column 2'-6" Outside Diameter
8"
8"
9 7/8"
2'-0"
Sub Soil
Top Soil
1'-9"
3"
Puddled Clay
1'-11" Diameter
24" Manhole
39'-3"
31'-3"
3'-0"
9"
3"
1'-0"
B
B
C
C
Top of Floor
Bottom of Floor
Minimum 3"
Sub Floor
10'-1 3/4"
10'-1 3/4"
10'-1 3/4"
10'-1 3/4"
20'-3 1/2"
20'-3 1/2"
20'-3 1/2"
20'-3 1/2"
VERTICAL SECTION
6"
4'-6"
4'-6"
6"
6"
9"
9"
4'-0"
4'-0"
5'-6"
5'-6"
7'-0"
9"
SECTION - B-B
SECTION - C-C

FIG. 7.—BALDWIN RESERVOIR: DETAILS OF FLOOR, COLUMNS, AND ROOF.

Conditions Assumed for Design.—Before the actual design of the various parts of the reservoir was begun, a careful study was made of the loads which might be imposed upon them during construction, as well as those which might be expected after the work was completed.

The conditions assumed for design before the completion of the reservoir, were:

1.—That the outside walls would be constructed and the back-fill placed against them before the roof arches were built.

2.—That the outside walls would be constructed and the roof arches built before any back-filling was placed against the walls or on the roof.

3.—That the concrete structure would be completed without the back-fill placed against the walls, or on the roof, and that either or both basins would be full of water.

The conditions assumed for design after the completion of the reservoir, were:

4.—That the concrete structure would be completed; that all the back-fill would be placed against the walls and on the roof; that there would be a uniform live load of 100 lb. per sq. ft.; but that there would be no water in either basin.

5.—That the same conditions would exist as are noted for Condition 4, except that either or both basins would be full of water.

In order to provide flexibility in operation and insure circulation of filtered water within the reservoir and also to detect leakage, it was assumed:

6.—That water would be delivered to either or both basins of the reservoir from the City's distribution mains prior to the completion of the Baldwin Filtration Plant.

7.—That water would be delivered to either or both basins of the reservoir from the filtration plant.

8.—That filtered water would be delivered to any or all of the distribution mains leading to the city and to the suction mains leading to the pumping stations from either or both of the basins.

9.—That means would be provided for emptying either basin for the purpose of cleaning or inspecting it and without disturbing the operation of the other basin.

In addition to the special features of design previously mentioned, careful consideration was given to circulation of the filtered water through each basin; to facilities for detecting leakage through the floor and lower sections of the walls; and to ample overflow capacity.

Reservoir Floor.—The floor of the reservoir slopes about 6 in. in 100 ft. from the north and south sides toward the center. The footings of all columns and stiffening walls were made separate from both the sub-floor and the finished floor. The sizes of the column footings were varied to suit the conditions of the shale, the smallest being 5½ ft. square, and the largest, 7 ft. square. In order to prevent the shale from disintegrating, and also to insure a uniform thickness for the finished floor, a sub-floor, having a minimum thickness of 3 in., was placed upon the shale immediately after the excavation was completed. No provision for expansion or contraction was made in the sub-floor. All irregularities in the excavation were filled with concrete.

The finished floor (Fig. 7) is 9 in. thick, and expansion joints are provided on the center lines of columns, namely, 20 ft. 3½ in. apart. The expansion joints consist of strips of prepared asphalt filler, 5 in. wide and ⅝ in. thick, placed at the bottom of the joints, the upper 4 in. being filled with hot asphalt. No steel was used in the finished floor except over the drains.

Columns.—In designing the connection between the tops of the round columns and the bottoms of the groined arches at their springing lines, a departure was made from usual engineering practice, in that the 2 ft. 6-in. round columns were flared out to a 2 ft. 6-in. square, which is the size of the groined arches at their springing lines. The ratio of length of column to outside diameter is 13.7 and the ratio of length of column to effective diameter is 15.75.

The maximum load on top of each column is about 184 000 lb., and the weight of each column is about 27 000 lb., making a total load of about 211 000 lb. on each column footing. Each column is reinforced with ten ¾-in. square twisted bars, placed 2 in. from the surface of the concrete. These bars are tied together with ⅜-in. round hoops, spaced 1 ft. on centers.

Groined Arch Roof.—Owing to the unusual depth of the reservoir, necessitating very long columns for supporting the roof, considerable throught was given to the most economical type of roof construction. Studies were made of several types of flat-slab design as well as of groined arches of various dimensions. These various designs were carefully compared as to cost, which led to the decision to use a groined arch roof with dimensions as shown in Fig. 7. The average thickness of concrete in the roof above the springing line of the groined arches is $8\frac{5}{16}$ in.

The method of calculating the stresses in the groined arches was based on the hypothesis of least crown thrust, as most generally used in theories of the voussoir arch. The total load on each arch was assumed as acting between vertical planes passed through the groins formed by the intersection of two adjacent arches, these planes intersecting at the center line of columns.

The line of thrust at the crown of the groined arches was assumed as acting through the upper third point. With this condition assumed, the least crown thrust to hold the groined arch in equilibrium under a maximum load of 2 ft. of fill and a live load of 100 lb. per sq. ft., produced a maximum compression of about 225 lb. per sq. in. in the concrete at a section about 6 ft. from the crown. The line of thrust at this point passes through the lower third point. The maximum shear is about 48 lb. per sq. in. under the same condition of loading. The upward pressure of the water did not affect the maximum compression in the arches, as this occurred at a point above the water level; but the maximum shear was slightly reduced.

Another feature in connection with the design of the roof to which special attention is called is the location of the stiffening walls (Fig. 8), which were provided to take the arch thrust in case of failure of a part of the roof. The two basins of the reservoir are each approximately 500 by 500 ft. and the two lines of stiffening walls in each basin divide the roof areas into approximately 250-ft. squares. The location of the stiffening walls does not interfere in any way with the circulation of the water across the basins. Their foundations and the two adjacent column foundations were designed to be poured monolithically, as were also the stiffening walls and the two adjacent columns.

Four ⅝-in. square bars were placed in the crowns of the arches, and sections containing not less than four groined arches were tied together by this steel. Construction joints in the roof were always placed at the crown, midway between columns. In order to prevent surface water from entering the reser-

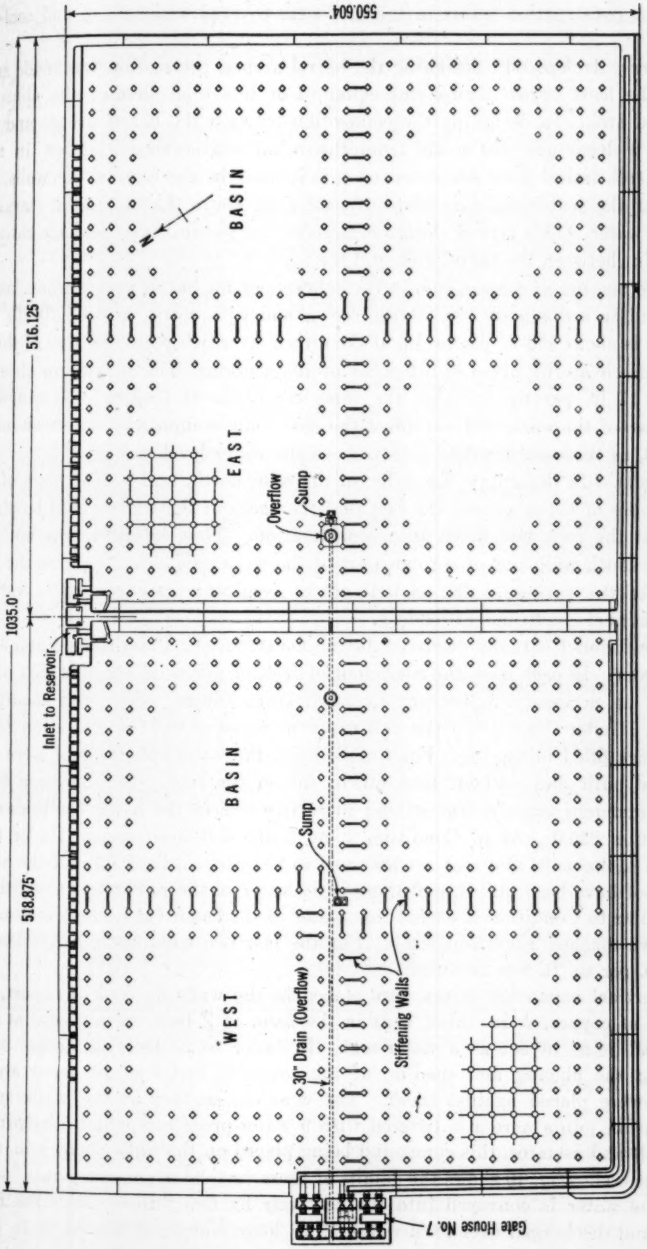

550.604'
516.125'
1035.0'
518.875'
Inlet to Reservoir
N
EAST BASIN
WEST BASIN
Overflow
Sump
Sump
30" Drain (Overflow)
Stiffening Walls
Gate House No. 7

FIG. 8.—PLAN OF FLOOR, BALDWIN RESERVOIR.

voir the construction joints in the roof were covered with burlap and coal-tar pitch.

Barrel Arches.—In designing the barrel arches, the section was made such that the least crown thrust was equal to or less than that of the abutting groined arch. In designing the connection between the barrel arches and the walls, a departure was made from the usual engineering practice, in that the barrel arches were supported on a seat built in the tops of the walls, the back of the seat being carried to a point 3 in. above the maximum elevation of the water. This type of design eliminates the possibility of leakage through the joint between the barrel arch and the wall.

The maximum compression in the concrete of the barrel arches when loaded with fill to a depth of 2 ft. at the crown and with a live load of 100 lb. per sq. ft., is about 130 lb. per sq. in. at the crown, at which point the line of thrust is assumed at the upper third point of the section. The maximum shear is about 17 lb. per sq. in. with the same condition of loading. The upward pressure of the water did not affect the maximum compression or shear in the arches, as these occurred at points above the water level.

Walls.—In designing the walls on the east, north, and greater part of the west sides of the reservoir, the fact that the backs of the walls would be placed against the rock was taken into consideration. This, however, was not true of the south wall, nor of a small part of the west wall. Fig. 9 shows the outlines of the various walls, as well as the resultant pressures under various hypothetical conditions of loading.

Condition 1 actually occurred during construction; Conditions 2 and 3 did not occur. In each case, the coefficient of friction given in Fig. 9 is that which would be necessary to prevent the wall from sliding under the conditions noted. Under Condition 1 the wall was considered as built to Elevation 229.25 and back-filled to the top. For Condition 2, the walls and the roof were considered built, but without back-fill, or fill on the roof. In Condition 3 the walls and roof are constructed and there is water in the basin (or basins) to Elevation 225.0. As in Condition 2, no filling has been done. Under Condition 4, the walls and roof are assumed to be built, all back-fill and the fill on the roof have been placed, and there is no water in the reservoir. Condition 5 is similar to Condition 4, except that water in the basin (or basins) is assumed to have reached Elevation 229.0. For the last two conditions, a live load of 100 lb. per sq. ft. was assumed.

Vertical expansion joints were placed in the walls 81 ft. 2 in. apart, and strips of copper, $\frac{1}{16}$ in. thick, bent in the form of Z-bars, were placed at each vertical joint to act as a water-seal. In order to protect the copper strips during the placing and spading of the concrete, strips of prepared asphalt filler were placed against them. The concrete surface of the walls at the expansion joints were also covered with a water-proof compound consisting of asphalt and asbestos, this compound being placed on the walls about $\frac{1}{2}$ in. thick by a trowel. Fig. 10 shows the details of these vertical expansion joints.

The water is conveyed into the reservoir by two flumes along the north wall and discharged over fixed weirs, 13 ft. long, which are spaced 40 ft. 7 in.,

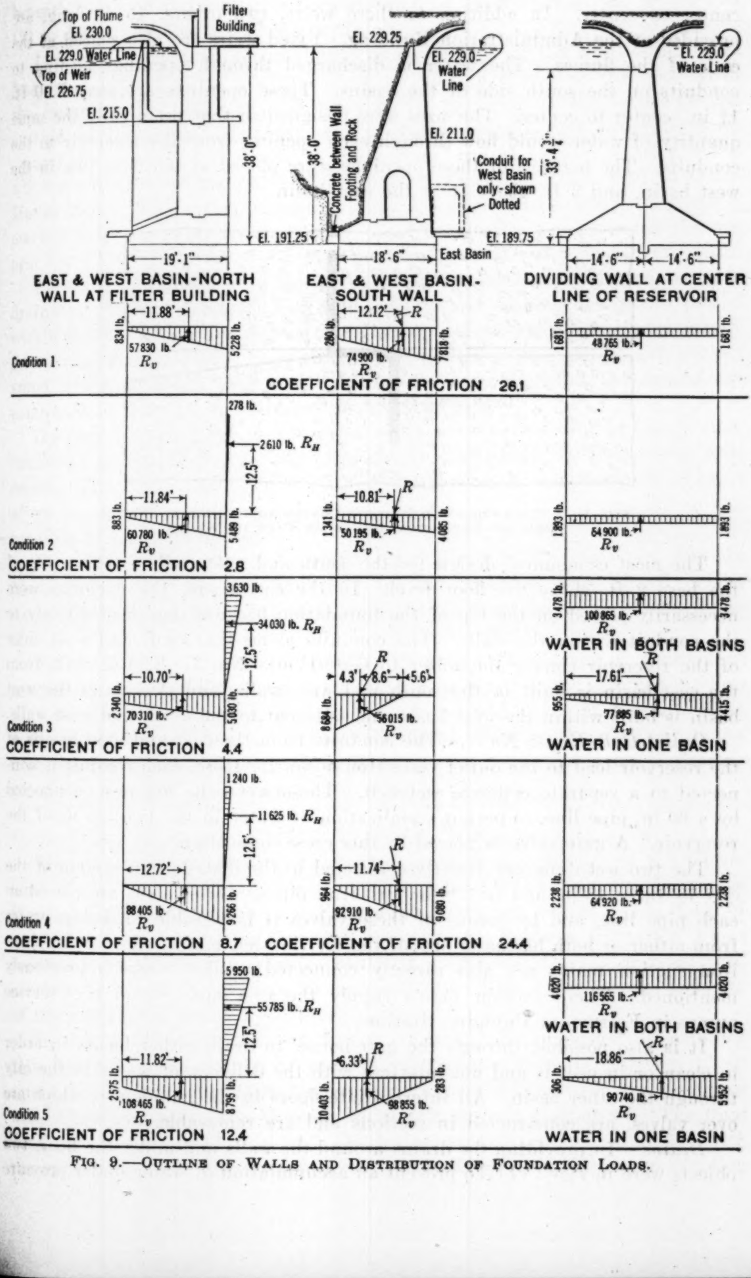

Top of Flume
El. 230.0
Filter Building
El. 229.0 Water Line
Top of Weir
El. 226.75
El. 215.0
38'-0"
19'-1"
EAST & WEST BASIN-NORTH WALL AT FILTER BUILDING
El. 229.25
El. 229.0 Water Line
38'-0"
Concrete between Wall Footing and Rock
El. 211.0
Conduit for West Basin only-shown Dotted
El. 191.25
18'-6"
East Basin
EAST & WEST BASIN-SOUTH WALL
El. 229.0 Water Line
33'-4½"
El. 189.75
14'-6"
14'-6"
DIVIDING WALL AT CENTER LINE OF RESERVOIR
Condition 1
11.88'
834 lb.
57830 lb.
R_v
5228 lb.
12.12'
R
280 lb.
74900 lb.
R_v
7818 lb.
1681 lb.
48765 lb.
R_v
1681 lb.
COEFFICIENT OF FRICTION 26.1
Condition 2
278 lb.
2610 lb. R_H
12.5'
11.84'
883 lb.
60780 lb.
R_v
5489 lb.
10.81'
R
1341 lb.
50195 lb.
R_v
4085 lb.
1893 lb.
64900 lb.
R_v
1893 lb.
COEFFICIENT OF FRICTION 2.8
Condition 3
3630 lb.
34030 lb. R_H
12.5'
10.70'
2340 lb.
70310 lb.
R_v
5030 lb.
R
4.3'
8.6'
5.6'
8684 lb.
56015 lb.
R_v
3478 lb.
100865 lb.
R_v
3478 lb.
WATER IN BOTH BASINS
17.61'
R
955 lb.
77885 lb.
R_v
1415 lb.
WATER IN ONE BASIN
COEFFICIENT OF FRICTION 4.4
Condition 4
1240 lb.
11625 lb. R_H
12.5'
12.72'
88405 lb.
R_v
9266 lb.
11.74'
R
964 lb.
92910 lb.
R_v
9080 lb.
2238 lb.
64920 lb.
R_v
2238 lb.
COEFFICIENT OF FRICTION 8.7
COEFFICIENT OF FRICTION 24.4
Condition 5
5950 lb.
55785 lb. R_H
12.5'
11.82'
2575 lb.
108465 lb.
R_v
8795 lb.
6.33'
R
10403 lb.
98855 lb.
R_v
283 lb.
4020 lb.
116565 lb.
R_v
4020 lb.
WATER IN BOTH BASINS
18.86'
R
306 lb.
90740 lb.
R_v
5952 lb.
WATER IN ONE BASIN
COEFFICIENT OF FRICTION 12.4

FIG. 9.—OUTLINE OF WALLS AND DISTRIBUTION OF FOUNDATION LOADS.

center to center. In addition to these weirs, two others, 15 ft. long. are provided at the Administration Building. Fixed weirs are also placed at the ends of the flumes. The water is discharged through openings leading to conduits on the south side of the basins. These openings are spaced 10 ft. 1¾ in., center to center. The sizes were designed so that practically the same quantity of water would flow through each opening from the reservoir to the conduits. The bottoms of these openings were placed at the floor line in the west basin, and 2 ft. above it in the east basin.

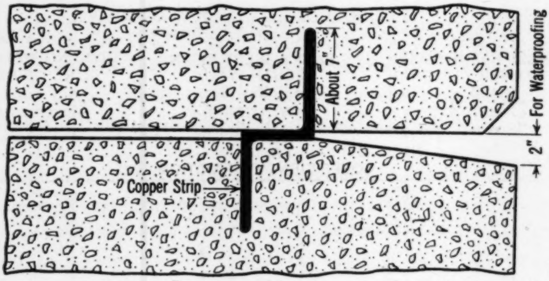


FIG. 10.—SUGGESTION FOR PLACING CONTINUOUS VERTICAL COPPER STRIP AT EXPANSION JOINTS ON FACE OF WALLS.

The most economical design for the south and west walls fixed the top of the base 2 ft. above the floor level. In the east basin, the openings were necessarily placed on the top of the foundation because they lead directly to the conduit within the walls. The conduits along the south and west sides of the reservoir convey the water to Gate-House No. 7. The conduit from the east basin is built in the south and west walls, and that from the west basin is built within the west basin and adjacent to the south and west walls.

Outlet Gate-House No. 7.—The conduits from the east and west basins of the reservoir lead to the outlet Gate-House No. 7. Here, each conduit is connected to a separate concrete wet-well. These wet-wells are cross-connected by a 60-in. pipe line, to permit equalization of water in the two basins of the reservoir. A gate-valve is placed in this cross-connection.

The two wet-wells are directly connected to the distribution system of the city by three 48-in. and two 36-in. cast-iron pipes. Gate-valves are placed on each pipe line, and by means of these valves it is possible to deliver water from either or both basins to any of the distribution mains. Two 48-in. cast-iron suction mains are also directly connected to the conduits previously mentioned. These suction mains supply the first and second high-service pumps in Fairmount Pumping Station.

It is also possible, through the gate-house, to drain either basin, in order to clean or inspect it and not interfere with the delivery of water to the city through the other basin. All intermediate floors in the gate-house, which are over valves, are constructed in sections and are removable.

Drains.—In providing the drains around the walls and under the floor, two objects were in view: (1) To prevent an accumulation of water under pressure beneath the floor and excessive accumulation of water back of the walls; and (2) to detect and locate leakage easily.

It is obvious that an accumulation of water back of the walls either from leakage or ground-water might produce an upward thrust on the floor when the water in the reservoir is lowered below the level of the water on the outside of the walls. Drains were provided beneath the reservoir floor, around the outside of the walls at the bottom, and at about Elevation 211.0. All these drains, except parts of the one at Elevation 211.0, lead to Gate-House No. 7, where they discharge into a sump. Each drain is properly marked in the gate-house, so that it would be possible to determine the approximate location of any leakage or accumulation of water behind the walls.

Particular attention is called to the detail in connection with the outside drains at the bottom of the east and north walls. These drains are placed within the walls, and are all cast-iron bell-and-spigot pipe with leaded joints. At varying intervals (not more than about 80 ft.) pockets, filled with broken stone, are provided in the rock backing of the walls, and the drains are connected to them by short pieces of cast-iron pipe.

Overflow.—Each basin is provided with an overflow and a sump, each connected to the 30-in. cast-iron pipe drain which leads to Fairmount Reservoir. The sump connections are for the purpose of emptying the basins of all water that cannot be taken out through the drains connected to the conduits at the gate-house.

Ventilation.—In order to provide ventilation for the reservoir, fifty-six 24-in., four 36-in., and two 48-in., ventilating manholes were placed in the roof. Those along the north side were placed over the weirs of the flume through which the water flows from the filtration plant.

Fill on Roof.—The concrete roof is covered with a layer of thoroughly compacted clay puddle, 6 in. thick at the crowns of the groined arches. This, in turn, is covered with 1 ft. of sub-soil and 6 in. of top-soil. The entire top of the reservoir is seeded with grass. Because the reservoir slopes from the north and south sides toward the center, it was necessary to provide a drain to carry off any accumulation of surface water.

Alum Storage Building

The alum storage building (Fig. 1) is located on a switch of the Cleveland Short Line Railroad, about ½ mile west of the Baldwin Filtration Plant and adjacent to the new Fairmount Pumping Station. The lack of railroad connection to the chemical house of the Baldwin Filtration Plant necessitated this separate storage building. This building is of reinforced concrete and is 51 ft. by 87 ft. in plan and 43 ft. high. The storage capacity of the building is about 1 000 tons.

In the building are four reinforced concrete bins supported on columns (see Fig. 11). The bottom of each bin terminates in three hoppers, 16 ft. square at the top with 15-in. square openings in the bottom, and equipped with single undercut gates through which the alum is discharged. The total depth of the bins is 17 ft. 6 in. The hopper sides have a slope of 50° with the horizontal. The bins extend across the building, and a common dividing wall separates one bin from another. The entire area under the bins is used for truck runways.

CONSTRUCTION

The construction of Baldwin Filtration Plant was carried on under general contracts for the different parts of the work. The construction operations will be described in the order in which they were done.

Baldwin Reservoir

In the winter of 1914-15, the City, by direct labor, began preliminary work by stripping the top-soil from the site of a proposed 130 000 000-gal. open reservoir. On April 9, 1915, a contract was awarded for the excavation of this reservoir. The contractor had completed about three-fourths of the work when he encountered financial difficulties and, as a result, suspended operations in July, 1918. All the excavation remaining to be done was in solid rock and, in consequence, the City entered into a "cost-plus" agreement with a quarrying company to complete the work. Excavation was resumed in the winter of 1918-19, and completed in 1920, except for final trimming.

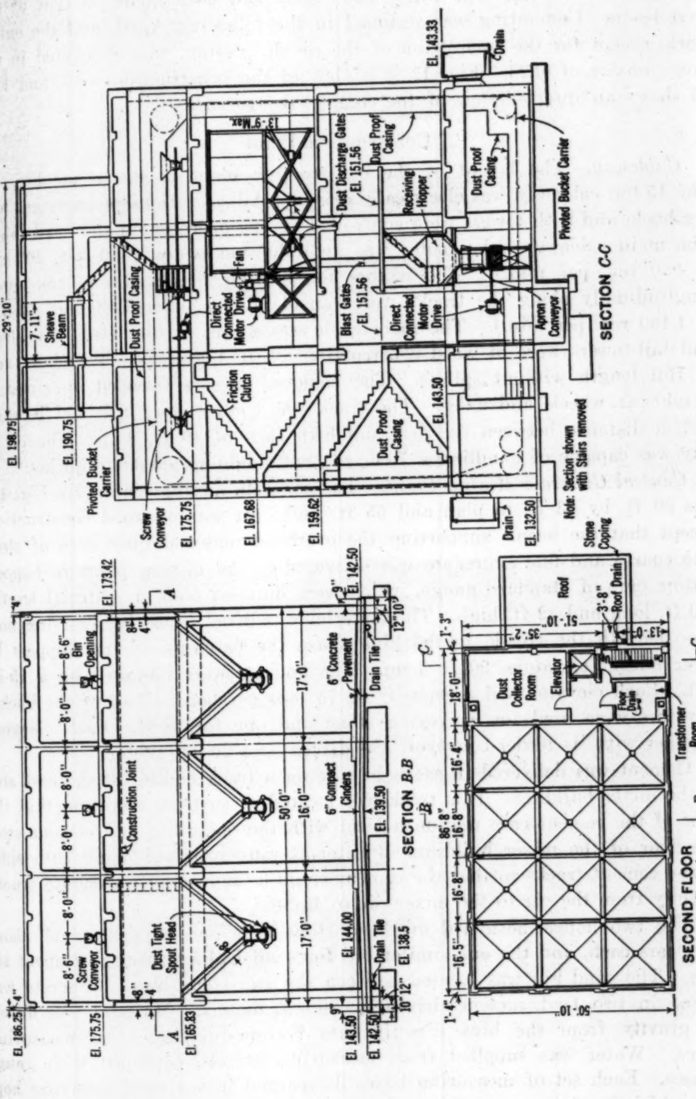
During the period covered by the excavation work, the location of the proposed second filtration plant, as explained under "History and Development of the Project", had been changed to the Baldwin site. Accordingly, additional land was purchased on either side of the reservoir, and this extended the area of city-owned land to the streets surrounding the site.

Bids were received and a contract for the construction of the reservoir was entered into on May 27, 1921; but objection was taken to this award and it was annulled by the Courts. The work was re-advertised, bids were received again, and the contract was awarded to the low bidder on November 18, 1921.

While the contractor was assembling his construction plant, he began the final trimming within the reservoir, and the final excavation for the wall footings. Due to the disintegration of the shale on exposure to air, it was necessary to leave the final trimming of all excavation until just prior to placing the concrete.

Pending the erection and completion of the central concrete mixing plant and the Lidgerwood 15-ton cableway, the contractor erected a temporary mixing plant within the reservoir excavation, near the south wall. Concrete mixed at this plant was hoisted to the top of a steel tower and distributed by chutes. By this means concreting of the south wall was commenced in the spring of 1922.

After the completion of the central mixing plant and the cableway, the temporary mixing plant was dismantled. After this, all concrete was mixed at the central mixing plant, and distributed by the movable cableway. The cable spanned the work in a north and south direction and was capable of movement in an east and west direction over the entire length of the reservoir. Work on the floor, column foundations, and columns, and on the groined arch roof, was started along the east wall and progressed toward the west.



SECOND FLOOR PLAN AT A-A
SECTION B-B
SECTION C-C
Dust Tight Spout Head
Screw Conveyor
Construction Joint
Bin Opening
6" Compact Cinders
6" Concrete Pavement
Drain Tile
Dust Collector Room
Elevator
Floor Drain
Transformer Room
Roof
Roof Drain
Stone Coping
Drain
Note: Section shown with Stairs removed
Pivoted Bucket Carrier
Dust Proof Casing
Sheave Beam
Friction Clutch
Direct Connected Motor Drive
Fan
Blast Gates
Dust Discharge Gates
Receiving Hopper
Apron Conveyor
El. 186.25
El. 175.75
El. 165.83
El. 143.50
El. 142.50
El. 144.00
El. 138.5
El. 139.50
El. 173.42
El. 198.75
El. 190.75
El. 167.68
El. 159.62
El. 132.50
El. 151.56
El. 143.33
50'-0"
86'-8"
29'-10"
7'-11"
13'-9" Max.
17'-0"
16'-0"
8'-6"
8'-0"
16'-5"
16'-8"
16'-4"
18'-0"
13'-0"
35'-2"
51'-10"
50'-10"
3'-8"
1'-3"
1'-4"
10'-12"
12'-10"

FIG. 11.—GENERAL SECTIONS, ALUM STORAGE BUILDING.

Work was continued night and day during 1922, until the winter weather stopped operations in December. At this time the entire roof of the east basin had been completed and a good start had been made on that of the west basin. Concreting was resumed in the following April, and the entire work, except for the completion of the rough grading, was completed in the late summer of 1923. Fig. 12 is a view of the construction work and Fig. 13 shows an interior view of the completed structure.

Construction Plant

Cableway.—The layout of the construction plant is shown on Fig. 14. The 15-ton cableway was electrically driven. All operations of carriage, lifting hook, and both towers were controlled by one operator on the head tower. The main cableway hoist motor was an induction motor (440-volt, 300-h.p., at 950 rev. per min.). The motors for moving the head and tail-towers longitudinally along the tracks were considerably smaller (440-volt, 75-h.p., at 1 150 rev. per min.). The main cable was 2½ in. in diameter. The head and tail-towers were of wood construction, 85 ft. high, and all timbers were of full length without splices. The cableway towers traveled on standard freight-car wheels and axles. The track was composed of five 100-lb. rails, with a distance between the first and fifth rails of 40 ft. 2 in. The cableway was capable of handling a 7-cu. yd., bottom-dump, concrete bucket.

Central Concrete Mixing Plant.—The concrete mixing plant (see Fig. 14) was 30 ft. by 63 ft. in plan and 55 ft. high. It was of wood construction, except that the beams supporting the overhead material bins were of steel. The coarse and fine aggregate was delivered to the mixing plant in hopper-bottom cars of standard gauge, which were dumped from a material trestle, 250 ft. long and 22 ft. high. The material was lifted from the receiving hoppers beneath the trestle to the bins above the batch-measuring hoppers by three bucket elevators, 90 ft. long. The buckets were mounted on a 15-in. belt. Each elevator had a capacity of 75 tons per hour. Two of the bucket elevators were used for conveying stone and one for sand. Each elevator, together with its feeder conveyor, was driven by a 20-h.p. motor.

Cement was delivered in sacks in cars on a track ending at the east side of the mixer building. This track was located at such an elevation that the floor of the cement cars was on a level with the operating and cement storage floor of the mixer building. Platforms were provided along both sides of the cement tracks so that the cement could be moved by warehouse trucks directly from the car to the mixers or to storage.

The two hopper-bottomed overhead stone bins had a capacity of about four cars each, and the one similar bin for sand had a capacity of about six cars. The sand bin was located between the two stone bins. Concrete was mixed in two 1-yd. mixers, driven by 30-h.p. motors. Material was drawn by gravity from the bins directly into bottom-dumping steel measuring boxes. Water was supplied from measuring barrels, equipped with gauge glasses. Each set of measuring boxes discharged into a steel receiving hopper, which, in turn, emptied into the body of the mixing drum. Cement was dumped from bags through a grating in the floor of the storage room directly

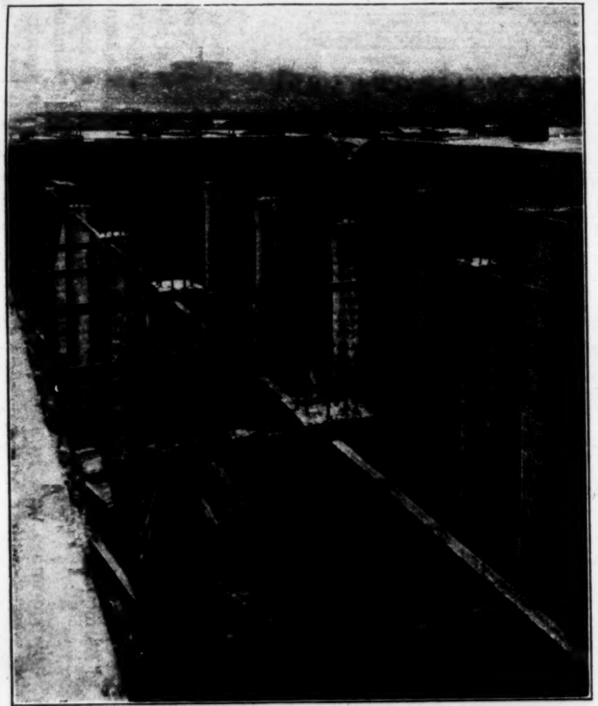


FIG. 12.—VIEW SHOWING BRACING AT TOPS OF CONCRETE COLUMNS, PREVIOUS TO PLACING FORMS FOR ARCHES, BALDWIN RESERVOIR.

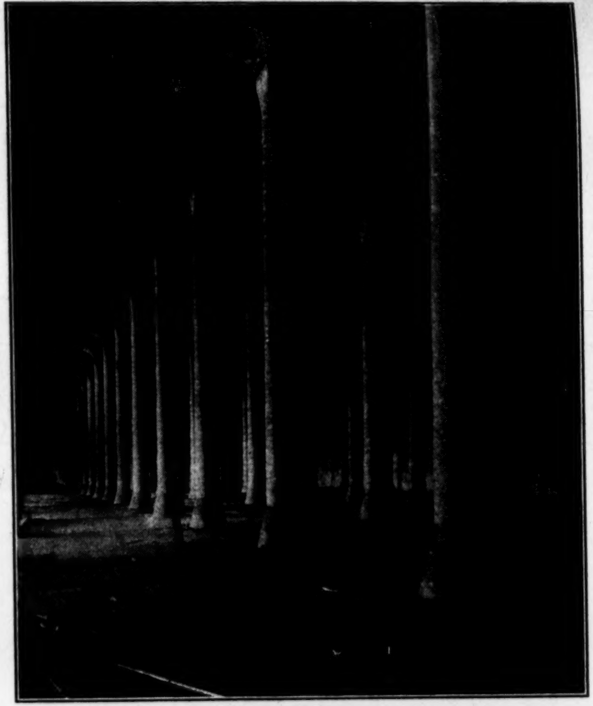


FIG. 13.—INTERIOR VIEW OF BALDWIN RESERVOIR.

into this receiving hopper. As a result of this arrangement, the sand, stone, cement, and a measured quantity of mixing water were supplied to the mixer simultaneously. The mixers were discharged into a common conical steel hopper with a capacity of about 9 cu. yd. A slide-gate at the bottom of this hopper permitted the concrete to be discharged into the 7-yd. concrete bucket as needed.

Placing Concrete.—The concrete was poured from 7-yd. buckets. After the buckets were filled they were transported on flat cars running on narrow-gauge (36-in.) tracks. Locomotives hauled the loaded cars to a position under the cableway, which had been previously moved so as to span over the form to be filled with concrete. An emptied bucket, hanging on the cableway hook, was placed on an empty flat car, and the full bucket of concrete was then picked up by the hook and moved to the proper position directly over the form and lowered to position for pouring. During the time that the bucket was being moved along the cable, the empty bucket was on its way to the mixer for refilling.

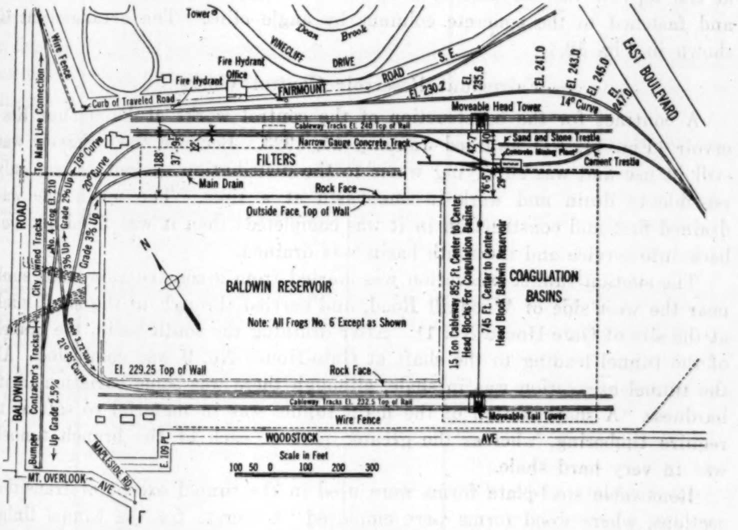

Tower
Doan Brook
VINECLIFF DRIVE
Fire Hydrant
Office
Fire Hydrant
FAIRMOUNT ROAD S.E.
Wire Fence
Curb of Traveled Road
El. 230.2
El. 235.6
El. 241.0
El. 243.0
El. 245.0
El. 247.0
EAST BOULEVARD
14° Curve
Moveable Head Tower
Cableway Tracks El. 240 Top of Rail
Narrow Gauge Concrete Track
Sand and Stone Trestle
Concrete Mixing Plant
Cement Trestle
To Main Line Connection
19° Curve
20° Curve
9½'
12'
37'
188'
42'-7"
40'
76'-5"
29'
FILTERS
Main Drain
Rock Face
Outside Face Top of Wall
ROAD
BALDWIN
City Owned Tracks
No. 4 Frog El. 210
3% Up
Grade 2% Up
Grade 3% Up
Contractor's Tracks
Grade 3.77% Up
21° 35' Curve
Up Grade 2.5%
Bumper
N
BALDWIN RESERVOIR
Note: All Frogs No. 6 Except as Shown
15 Ton Cableway 852 Ft. Center to Center Head Blocks For Coagulation Basins
745 Ft. Center to Center Head Block Baldwin Reservoir
COAGULATION BASINS
El. 229.25 Top of Wall
Rock Face
Cableway Tracks El. 232.5 Top of Rail
Wire Fence
Moveable Tail Tower
WOODSTOCK AVE
E. 109 PL.
MAPLESIDE RD.
MT. OVERLOOK AVE
Scale in Feet
100 50 0 100 200 300

FIG. 14.—COAGULATION BASINS AND FILTERS, CONSTRUCTION PLANT LAYOUT.

Forms for Concrete.—Considerable study was given to the design of forms, in order to keep costs as low as possible. Practically all forms were made in separate panels so as to permit their use many times. Wall-form panels were made of various sizes, depending on their intended use. The largest single panels were about 20 ft. high and 45 ft. long. Forms for the groined arch roof were made in four sections for the part supported by one column.

Except for columns, all forms were made of ⅞-in. center-matched lumber, nailed to 2 by 6-in. studs which were spaced on 16-in. centers. Vertical forms were braced by 4 by 6-in. timbers running horizontally outside the studding and held from spreading under the force of the horizontal thrust of the wet concrete, by $\frac{5}{8}$-in. bolts running through the section to be concreted. These bolts were made up with pump-rod couplings, spaced 4 in. inside of each form face. These horizontal timbers were spaced a variable distance apart, depending on their elevation above the bottom of the form. Column forms were made of steel, each complete form consisting of two halves. All forms were handled by the cableway.

Due to the great depth of the reservoir, unusual methods were used in supporting the forms for the groined arch roof. The entire load of the roof forms as well as of the concrete, was carried by wood and steel collars fastened around the tops of the previously poured columns, and by eight wooden posts, each 8 in. square, extending from floor to roof, and located around the perimeter of the groined arch unit supported by one column. One post was located at each of the four corners, and additional posts midway between the corner posts. These posts were braced in a horizontal plane at the tops of the columns by 4 by 6-in. timbers, extending between posts and fastened to the concrete columns by angle clips. The arrangement is shown in Fig. 12.

Fairmount Reservoir Control Works

A contract for the construction of the control works at Fairmount Reservoir (Fig. 1), was awarded January 15, 1923. Because the reservoir was still in use and was supplying water to the distribution system, it was only possible to drain and work in one basin at a time. The north one was drained first and construction in it was completed; then it was filled and put back into service and the south basin was drained.

The suction tunnel excavation was started from a construction shaft sunk near the west side of Woodhill Road, and carried through to the shaft sunk at the site of Gate-House No. 11. After draining the south basin, the branch of the tunnel leading to the shaft at Gate-House No. 9 was completed. All the tunnel excavation was in shale, although there was some variation in its hardness. A short section of the main tunnel was in material so soft as to require timbering, whereas the greater part of each of the branch tunnels was in very hard shale.

Removable steel-plate forms were used in the tunnel except at transition sections, where wood forms were employed. Concrete for the tunnel lining was mixed at the top of the construction shaft in a drum mixer and was transported to the point of placing by a pneumatic concrete blower, which forced the mixed concrete through a 4-in. steel pipe line.

Coagulation Basins

Excavation.—The excavation for the coagulation basins was done under a contract, bids for which were received April 29, 1920. The contractor began work June 25 of the same year and continued working until March 8, 1921. The area covered by the basins was excavated to an elevation 1 ft. above the base of the proposed wall foundations. The material encountered was clay and soft shale. Steam shovels loaded the excavated material into dump trucks and dump wagons, which removed it to storage and disposal piles.

Three steam shovels were used regularly on the work. The maximum number of dump trucks used in any one day was twenty-three and the maximum number of teams was thirty. Progress of the work was greatly delayed in wet weather, due to the inability of the trucks and teams to maneuver in the wet clay.

The total material excavated was approximately 117 000 cu. yd. About 20 000 cu. yd. suitable for back-filling were hauled to a storage pile across Doan Brook and about 1¼ miles east of the site of the work. An additional 10 000 cu. yd. were used in completing a fill along the south side of Doan Brook, and in the landscaping plans in the development of the valley of this brook. An additional 30 000 cu. yd., suitable for back-filling, as well as about 55 000 cu. yd. intended for use as clay puddle on the roof of the filtered-water reservoir, were stored between the site of the basins and the boulevard to the east.

Due to delay on the part of the City in acquiring some of the land needed for the site of the filters—the general excavation of which was also included in this contract—it was necessary to postpone the excavation of the depressed portions of the four coagulation basins, as well as the excavation for the filters.

Construction.—A contract for the construction of the coagulation basins was awarded June 1, 1923. The contractor who was just completing the Baldwin Reservoir, was the low bidder on the contract for this work. This made possible, by some re-arrangement, the use of much of the old construction plant on the new contract. (See Fig. 14.)

Because the coagulation basins were located along the east side of the reservoir and extended some distance beyond its north and south limits, a direct extension eastward of the existing cableway tracks would bring them inside the north and south ends of the basins. The contractor accordingly moved the head-tower tracks to a new position, parallel to the old, and north of the existing concrete mixing plant, and extended the track to the easterly limit of the coagulation basins. The mixing plant had been purposely located just outside the north limit of excavation. Insufficient room for the tail-tower tracks, between the south wall of the basins and Woodstock Avenue, prevented the location of these tracks so that the cableway carriage could span the full length of the basins. However, the existing tail-tower tracks were extended east over the south end of the basins for their full width. This arrangement placed the entire coagulation basin area, except the part at the southerly end, within working position of the hoisting hook on the cableway carriage. The increased span necessitated the substitution of a 5-cu. yd. bucket in the place of the 7-cu. yd. bucket used on the reservoir, thus reducing the capacity of the cableway to about 12 tons.

After completing the re-arrangement of the construction plant, the contractor excavated for and placed the concrete foundation for a considerable length of the by-pass conduit. At the same time a ¾-yd. caterpillar steam shovel was used in excavating the depressed portion of Basin No. 1. When considerable of this basin had been excavated to within a few inches of the final grade, a steam channeling machine was set up over the location of the longitudinal drain, and channel cuts were made along each side. After excavating, this method left the walls of the drain in good condition to receive the concrete lining, using only interior forms and depositing the concrete directly against the shale through which the channel cuts had been made. This practice of making cuts through the shale and rock and depositing concrete directly against the vertical surface was typical for similar conditions throughout the work. It was of particular value in situations like the one before mentioned, where no back-filling could be permitted behind the drain walls on account of possible settlement and resultant cracking of the basin floor.

At the same time that the longitudinal drain was being built, foundations for the dividing wall between Basins Nos. 1 and 2 and the column foundations for Basin No. 1 were constructed. After concreting the drain walls, dividing wall, and column footings for a sufficient length of basin, the last few inches of shale were removed from the general floor area and 4 in. of sub-floor concrete were placed. This concrete formed a working base on which the steel for the finished floor could be placed, and insured that the full depth of concrete for this floor would be free from disintegrated shale. If the sub-floor had been omitted, it would have been possible for serious disintegration of the shale to have occurred during the placing of the floor steel. This shale might have worked up into the floor concrete while it was being placed, thus forming weak spots.

At all construction joints in the floor-slab, as well as where the floor concrete joined the column and wall foundations, particular care was taken to insure water-tight joints. Before pouring the floor, strips of prepared asphalt filler ⅜ in. thick were placed at the bottom of the joints. These extended to within 4 in. of the floor surface. In the upper part of the joint, flat steel plates were placed. They were withdrawn after the concrete had set, and the space was filled with melted asphalt. This type of joint was used throughout the work in locations requiring a water-tight floor joint. In addition, all vertical joints in the floor and sub-floor were staggered at least 12 in.

At the transverse joints in the wall and conduit foundations, copper strips provided adequate water-stops, and the poured asphalt joints were not required. In order to provide room for expansion at these joints, the existing concrete surface was coated with about ½-in. layer of an asphaltic fiber cement. This provided a cushion, which could be compressed by the expanding concrete and would prevent spalling of the concrete edges or displacing of the sections due to temperature changes. At vertical construction joints in the walls subjected to water pressure, a copper strip—attached at its lower end to the horizontal copper strip in the foundation—served as a water-stop, and provisions, the same as previously outlined, were made to take care of temperature changes.

Only a few sections of the finished floor in Basin No. 1 and even less in Basin No. 2 had been completed, when concreting was stopped for the winter of 1923-24. The remainder of the year 1923, as well as the first few months of 1924, were used in completing the excavation of the depressed part of Basin No. 1, as well as the depressed parts of the other three basins. This excavated material was loaded on standard gauge dump cars and placed in a railroad fill some distance from the site of the work.

Early in the spring of 1924, the contractor started making forms for the columns and for the groined arch roof of the basins. (See Fig. 15.) For the sections for the arched roof 596 forms were made, or enough to permit pouring the arches over 149 columns at one time. This required the use of each form on an average of seven times during the course of the work.

Concreting in the groined arch roof was commenced in Basin No. 1 on April 26, 1924. The columns were poured for the full height in one operation, and then, before commencing on the arches, a sufficient time was allowed to elapse to permit the concrete in the column forms to shrink. The arches were poured in groups of four, and usually three of these groups were completed in one pouring, making a double row of arches from one barrel arch to the next. Column and arch forms were left in place 14 days in the spring and 10 days in the hot summer weather. The arches were partly loaded after 21 days.

On November 1, 1924, the work in Basin No. 4 had been completed as far south as the running track in front of the tail-tower. The tail-tower tracks were then removed from the easterly half of this basin and construction work in this area carried on by means of a 35-ton locomotive crane, operating from the running track in front of the tail-tower tracks, and also from the track just outside the south wall. As soon as the eastern half of this end of the basin had been completed, the remaining tail-tower tracks in this basin were removed and operations carried on by a locomotive crane, the same as for the eastern half of the basin. In this same manner the other basins were completed until the eastward travel of the cableway was stopped at the westerly side of Gate-House No. 1.

Filters and Effluent Galleries

Excavation.—The general excavation of the Administration and Filter Building site was included with the excavation for the coagulation basins, for which bids were received April 29, 1920. Delay by the City in acquiring the land on which these structures were to be located prevented the immediate excavation, and the work was assigned to the contractor having the general construction of Baldwin Reservoir. Excavation of this site was begun in the late summer of 1923.

Construction.—On October 24, 1923, bids were received for the construction of the Administration and Filter Building, and the contract was awarded to the low bidder, who was already engaged in constructing the coagulation basins.

By extending the cableway tracks west (Fig. 14) for the full length of the filter building, the same plant could be used as for the coagulation basins. The contractor immediately began excavating the 12-in. cover left on the foundation bed and prosecuted the work so vigorously that, when concrete operations were suspended near the end of December, 1923, the concrete floor-slab for the southeast effluent gallery, as well as two-thirds of the walls around this same section, were completed.

During the remainder of the winter, the excavation of the main drain was completed. The material encountered was composed of layers of a hard, fine-grained sandstone, between which were thin layers of hard shale. The sides of the main drain were cut with a channeling machine, and the lateral connections, located at every filter, were cut by drilling closely spaced holes, with jack-hammers.

Concreting was resumed early in the spring of 1924. The roof-slab over each effluent gallery was poured in five sections. The construction joints were placed at the dividing plane between the double beams located at the end of every second filter. Due to the length of time required in pouring each roof section of the effluent gallery and the great amount of beam steel located just above the columns, it was found best to pour all the columns in any one section as soon as the column and beam forms were completed, but before the beam and slab steel had been placed. The columns were poured up to the under side of the deepest beam supported by that column. Before placing the forms and concreting the last roof section in each gallery, all the pipe and fittings for the collector system were lowered into the gallery.

Each of the concrete filter tanks (Fig. 16), including the floor, walls, and top walks, was constructed in one operation. Seven sets of forms, and four additional sets, complete except for the outside forms for the two side walls, were prepared. This permitted work on eleven filter tanks at one time, as each of the four sets of forms lacking the outside wall forms, could be used between two filter tanks already poured and stripped.

Each filter tank was poured as a monolith, and the first step was to pour the floor. This served to hold the mortar in the concrete in the side walls and prevented its escape under the lower edge of the inner supported wall form.

The quantity of concrete in one filter tank was 156 cu. yd. The time required to concrete one filter ranged from 3½ to 5 hours, the general average being a little more than 4 hours. The first filter was concreted on May 12, and the last one on August 29, 1924.

CHEMICAL HOUSE AND MIXING FLUMES

Bids for constructing the chemical house and mixing flumes were received on July 23, 1924. These were rejected, the work was re-advertised, and new bids were received on August 20. The contract was awarded to the contractor already engaged in building the remainder of the plant.

Because the site of the work was partly covered by the existing cableway tail-tower tracks, it was necessary to restrict the westerly movement of the cableway to a point about midway of the work under this contract. The cableway tracks were then removed west of this point and excavation for the chemical house was begun.

Standard-gauge railroad tracks were laid along either side of the building site and locomotive cranes operating on these tracks were used to hoist

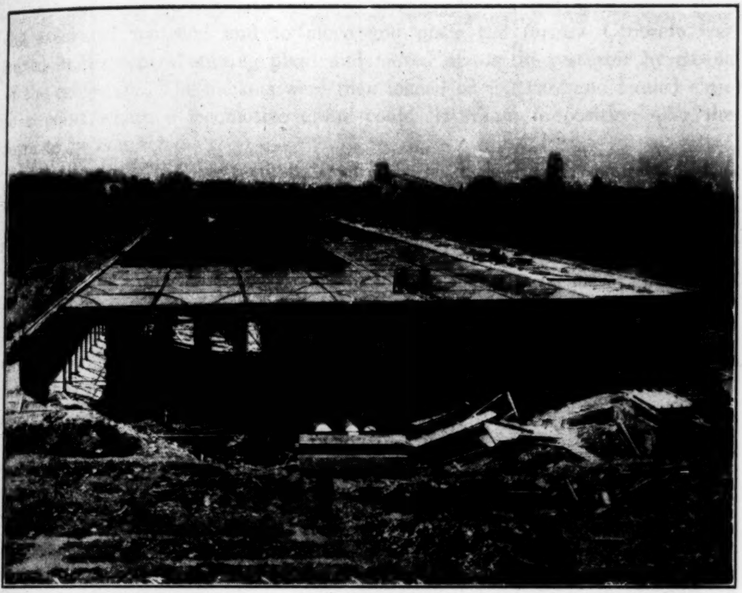


FIG. 15.—VIEW SHOWING FORMS FOR GROINED ARCH ROOF OF COAGULATION BASIN, BALDWIN FILTRATION PLANT.

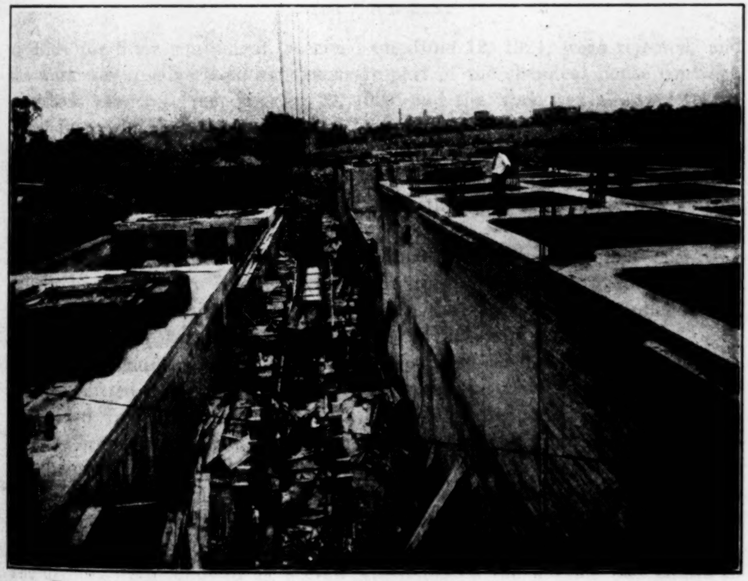


FIG. 16.—PIPE GALLERY AND FILTER TANKS DURING CONSTRUCTION, BALDWIN FILTRATION PLANT.

the excavated material and to move and place the forms. Concrete was mixed in the central mixing plant and moved across the reservoir by means of the cableway. The buckets were then loaded on flat cars and hauled close to a point where a locomotive crane could lift them to position over the form to be concreted.

By the time the westerly half of this work had been completed, the cableway was dismantled and a track was laid from the mixing plant around the east and south sides of the coagulation basins to connect with the working track on the south side of the chemical house. Concrete for the remainder of this work was brought from the central mixing plant by means of this track. The dissolving and solution tanks, as well as the small orifice boxes, were each poured in one operation up to a construction joint above the level to which they would be filled. This eliminated the possibility of leakage, which might have occurred, had there been a construction joint where the wall and floor joined.

To guard against the disintegration of concrete in the tanks to be used for alum solution, a richer mixture was used in order to obtain as dense a concrete as possible. For the dissolving and solution tanks, with 12-in. walls, concrete proportions of approximately 1:1.5:3 were used, and for the orifice tanks, with walls 6 in. thick, a mixture of approximately 1:1.25:2.5 was used. In addition, the interior surfaces of all these tanks were coated with a special acid-resisting paint.

FILTER EQUIPMENT

Bids for filter equipment, received on June 12, 1924, were rejected, and the work was re-advertised as a separate part of the chemical house contract. New bids were received on July 23, 1924, and the work was awarded to the low bidder, who immediately started manufacturing the equipment.

The cast-iron manifolds and laterals, as well as the effluent piping and rate controllers, were delivered to the site of the work in railroad cars. Delivery of the rate controllers, as well as of the filter manifolds, was started on October 8, 1924.

By January 30, 1925, the superstructure over the filters and operating gallery had so nearly reached completion that the contractor for the filter equipment was permitted to start placing the filter rate controllers. On February 10, the first manifolds were placed in position in the filters.

The large number of filters and the repetition of the same steps at each filter permitted doing the work in a very rapid and efficient manner. It was subdivided into different operations, each of which was performed by a separate crew. As an instance of this subdivision, the work inside the filters may be cited. One crew confined its efforts to placing in position and bolting down the manifolds, and then moving on to the next filter as soon as this operation was finished. Another crew followed and drilled the holes in the floor by means of which the lateral chairs were anchored in position. The drillers were followed by a crew which placed the laterals and chairs in position, and this work was followed by the various operations, such as pouring the lead joints, caulking joints, placing brass strainers on the manifolds, placing the various layers of gravel, and, finally, placing the filter sand.

The filter gravel and sand were purchased by contract. The first bids were rejected and new bids were received November 6, 1924. The coarse filter gravel, ranging from 1½ in. to ¾ in. in size, was obtained from the Ohio River, near East Liverpool. The gravel ranging from ¾ to ¼ in. in size, a part of that ranging from ¼ to ⅛ in. in size, as well as part of the gravel ranging from ⅛ to 1/16 in. in size, was obtained from Copley, Ohio. The remainder, as well as the filter sand, came from Phalanx, Ohio.

All gravel and sand was received in railroad cars on tracks laid close to the filter building. The three larger sizes of gravel were unloaded by shoveling on to a belt conveyor, which carried it inside the building and dumped it into a hopper from which it could be drawn into wheel-barrows. The smallest size of gravel, that from ⅛ to 1/16 in., and all the sand were unloaded directly from box cars into the filters by means of ejectors placed in the cars.

The following approximate quantities will give some idea of the amount of work involved in constructing the Baldwin Filtration Plant:

Excavation, in cubic yards	1 100 000
Fill, in cubic yards	290 000
Concrete, in cubic yards	185 000
Barrels of cement used	290 000
Brick	2 300 000
Cut stone, in cubic feet	102 000
Reinforcing steel, in pounds	10 900 000
Structural and miscellaneous steel, in pounds	3 000 000
Cast-iron pipe and fittings, in pounds	8 000 000
Filter sand, in cubic yards	5 400
Filter gravel, in cubic yards	3 500

These quantities do not include work on the alum storage building, the Fairmount Control Works, the pipe lines leading to the filter plant, nor the pipe lines leading from Gate-House No. 7, that connect into the distribution system. Fig. 17 is a panoramic view of the finished plant.

GENERAL EQUIPMENT

The equipment of the Baldwin Filtration Plant is best described under six main divisions: The alum storage plant, chemical house, administration and filter building, and lighting, heating, and telephone systems.

Alum Storage Plant.—To facilitate the unloading of alum from the box cars in which it is received, a power shovel or car plow, operated by a 5-h.p. motor at 1 200 rev. per min., is provided. The alum received from the car passes through a chute to an apron feeder conveyor driven by a 2-h.p. motor at 1 200 rev. per min. This conveyor can deliver the alum either to a 24 by 20-in. rigid hammer crusher, operated by a 20-h.p. motor at 900 rev. per min., or directly to a pivoted bucket carrier. The crusher has a capacity of 20 tons per hour, when crushing 5-in. lump alum to approximately ½-in. pieces.

The alum is lifted to the top floor of the building by a pivoted bucket carrier traveling at the rate of 40 ft. per min., and driven by a 5-h.p. motor at 1 200 rev. per min. The buckets are 18 by 15 in. and, after moving

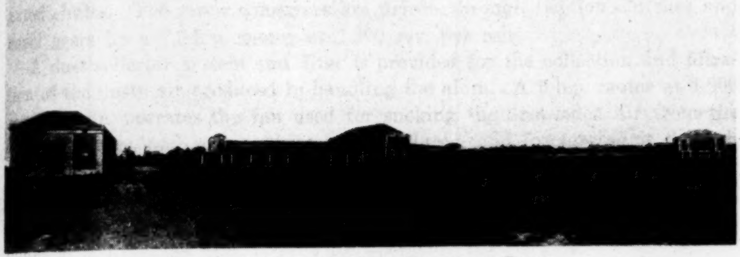


FIG. 17.—VIEW OF BALDWIN FILTRATION PLANT.

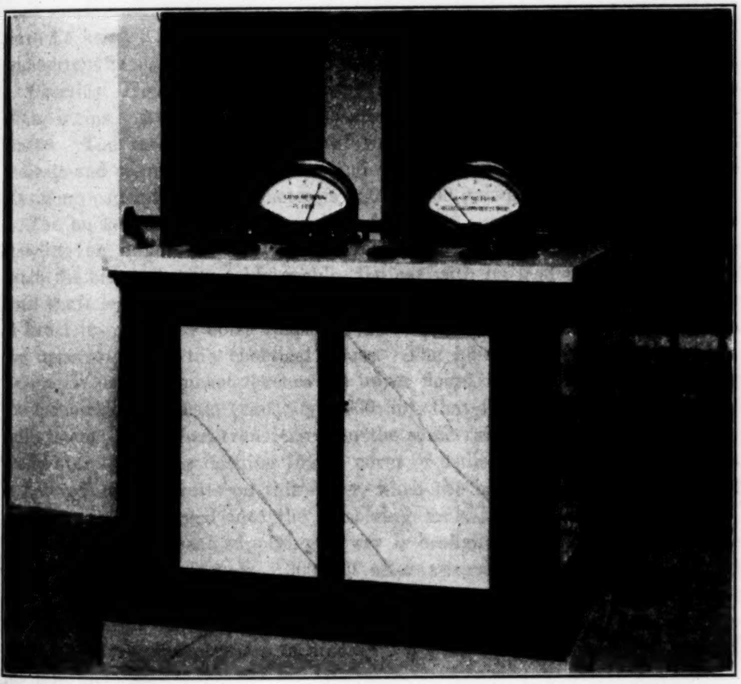


FIG. 18.—ONE OF THE BRONZE AND MARBLE FILTER OPERATING TABLES, BALDWIN FILTRATION PLANT.

horizontally with their load of alum, rise vertically to the upper floor, where they may feed any one of three 12-in. screw conveyors. These latter, each of which is 64 ft. in length, deliver the alum to the various bins through gated chutes. The screw conveyors are driven through friction clutches and bevel gears by a 7.5-h.p. motor at 1 200 rev. per min.

A dust collector system and filter is provided for the collection and filtration of the dusty air produced in handling the alum. A 5-h.p. motor at 1 200 rev. per min. operates the fan used for sucking the dust-laden air from the crusher, chutes, and rooms where it is produced, and for forcing it through the cloth filter.

The building is provided with a platform elevator, 5 ft. square, with a capacity of 2 500 lb. when moving at a speed of 150 ft. per min. A 13-h.p. motor at 900 rev. per min. is used for operating the elevator.

Electric current for operating the motors is 3-phase, 60-cycle, alternating current at 2 300 volts, which is reduced to 220 volts through three 15 kv-a transformers. A remote-control panel board with switches for operating the car plow, crusher, bucket carrier, screw conveyors, etc., is placed outside the room containing the power transformers. This same room also contains three 1.5 kv-a., 2 300 to 110-volt transformers for the lighting system. Electric current is supplied from generators in the Fairmount Pumping Station.

Chemical House.—The raw water entering this building through two 60-in. mains, passes through two Venturi tubes with throats 30 in. in diameter. The rate and total flow of water are shown on two meters, that indicate and record the rate of flow and also register the total flow. The rated maximum capacity of each meter is 120 000 000 gal. per day.

The bucket used to lift the alum from the storage bins, is filled while standing on a flat car, which runs on a narrow-gauge (24-in.) track beneath the bins. A turn-table enables the car with its load to be run into the hoist shaft leading to the top floor of the chemical house. The loaded bucket is lifted by a geared hoist (capacity 3 000 lb.) mounted on a mono-rail in the upper story of the chemical house. The hoist is operated by a 3-h.p. motor. When the bucket reaches the upper floors, it is moved by hand along the mono-rail to a scale (capacity 4 000 lb.), that forms a part of the mono-rail system. It is then transferred on the same rail system to a position directly over a circular opening in the cover of a dissolving tank. The bucket is lowered until it rests on this cover, when the bottom gate is opened, and the alum is discharged into the dissolving tank.

The electric current supplying power is brought into the building in the form of a 3-phase, 60-cycle, 2 300-volt alternating current, which is reduced to 220 volts through three 10 kv-a. transformers. Three 5 kv-a. transformers are also used to convert the 2 300-volt current to 110 volts for lighting purposes. The transformers are located in a separate room especially provided for this purpose.

The electric passenger elevator, in addition to being a convenience for the operating force, may serve as a means of carrying alum up from the bins to the dissolving tanks should the bucket hoist fail to operate. It has a lift of 56 ft., a capacity of 2 500 lb., and a maximum hoist speed of 150 ft.

per min., under full load. The shaft in which it operates is 6 ft. by 6 ft. 6 in. in plan. The elevator is driven by a 13-h.p. motor operating at 900 rev. per min.

The dust-collecting system consists of seven hoods, one over each possible loading position of the bucket. It also includes collector piping and valves and a dust arrester with a discharge hopper and outlet equipped with a dust valve and canvas spout. The fan is driven by a 10-h.p. electric motor running at 1 160 rev. per min.

The tanks for dissolving sulfate of alumina are each 6 ft. 6 in. by 5 ft. in plan and have a depth of 6 ft. 9 in. There is a 3-in. ledge 1 ft. above the bottom, which supports a wooden grid. This, in turn, supports a removable acid bronze screen upon which the alum is dumped. Below the bottom of the wooden grid, is a lead pipe distribution system for furnishing the dissolving water. In a recess at the end of the dissolving tank is a wooden baffle in line with the concrete wall below. Beyond that is an acid bronze screen, designed to remove trash of all kinds from the solution before it passes into the storage tanks. The wooden grid is made in three sections and is composed of 1⅝-in. by 3⅝-in. cypress joists set on edge with ⅞-in. spacer blocks between the joists. The joists of each section are bolted together with ¼-in. bolts painted with an acid-proof paint before assembling. The bronze screens are set in cypress frames, and are composed of acid-proof bronze wire, 0.08 in. in diameter, 4 meshes per in. for the strainer screen and 6 meshes per in. for the screen resting on the wooden grid. These dissolving tanks were designed for a maximum charge of 4 000 lb. of chemicals.

Each solution storage tank is 15 ft. 4½ in. by 18 ft. 3 in. in plan, with an average depth of approximately 10 ft. The bottom of the tank slopes 1 ft. 6 in. in its length, with a drain at the low point. The solution outlet from the tank is 19 in. above the drain. This design was adopted to form a sump into which trash or separated aluminum hydrate might settle and not be carried into the chemical pipe lines. To effect a mixing action in the solution tanks by driving water up through a grid at the bottom and supplying make-up water for bringing the solution to a predetermined strength, four 1½-in. lead pipes spaced equi-distant across the bottom of the solution tank and tied into a 2-in. header are provided and are supplied with water through a 2-in. feed line. The four 1½-in. laterals in the bottom of the tank are each provided with ¼-in. holes on 3-in. centers drilled in the top of the pipe.

Each orifice box is 1 ft. 9 in. by 4 ft. in plan and 3 ft. 9 in. deep, with the center line of the orifice set 1 ft. 11 in. above the bottom. Each box is divided into two compartments by a wooden baffle. Into one of the compartments the alum solution is discharged and passes over the baffle into the second compartment. The baffle acts to still the disturbance produced by the discharge of the solution. The float for maintaining a constant head over the orifice is placed in the second compartment. The orifice itself is placed in the wall of the box forming one end of this compartment.

The orifice boxes are designed in pairs with a common dividing wall. As the solution from either orifice leaves this box it passes into a common make-up box, 18 in. wide, extending across the full width of the two orifice boxes. A 1-in. water line, discharging into this make-up box, permits an increase in the volume of solution which assists in better distribution as it is fed into the water.

The orifice itself has a fixed width of ¾ in. and is capable of being varied from a minimum length of 2½ in. to a maximum of 7 in. The orifice plate is made of acid bronze and consists of a base plate set in the concrete to which the orifice plate proper is attached. One end of the orifice is designed to remain stationary while the other end is moved by a screw operated by a hand wheel. A pointer on the movable part of the plate indicates the opening on a scale set below.

For conveying the alum solution from the orifice boxes, a pipe leads from each set to a distributing trough over the rising well. This is a wooden trough, 70 ft. long, covered with sheet lead ³⁄₃₂ in. thick, and hung from bolts fastened to inserts placed in the concrete floor above.

The joints of the lead sheets are burned together, and on the down-stream side of the trough lead lips are spaced 2 ft. 6 in. apart. The trough is V-shaped, with a bottom angle of 90 degrees. It is 18 in. wide across the top and has a depth of 7 in. above the top of the fillet.

In order to obtain a uniform discharge of the solution from each lip, the level of the trough may be accurately adjusted by turnbuckles.

Administration and Filter Building.—There are four wash-water pumps on the floor at Elevation 216.0 for lifting the water from the effluent galleries to the wash-water tanks, in the top story of the Administration Building. They are operated through electric control actuated by means of floats in the wash-water tanks. The float contacts are arranged so that if the level of the water in the tanks continues to drop, additional pumps are thrown successively into service. The electric control panels are in the pump-room adjacent to the pumps. Push-button control is also provided in the pump-room. Two pumps, each with a capacity of 3 450 gal. per min., are driven by 75-h.p., induction motors of the squirrel-cage type, running at 1 760 rev. per min. Each of the other two pumps has a capacity of 1 850 gal. per min. and is driven by a 40-h.p. motor of the same type, running at a speed of 1 750 rev. per min. The current used in the motors for these pumps is a 3-phase, 60-cycle, at a potential of 2 300 volts.

Electric power for operating the motors on these pumps is obtained from two sources: From cables of the Municipal Electric Light Plant; and from the generators at the Fairmount Pumping Station.

The transformer and switchboard room contains the four primary control panels for the four wash-water pumps; three 25-kv-a. transformers for converting the current from 2 300 volts to 220 volts, for general power purposes other than that required for the wash-water pumps; and three 25 kv-a. light transformers for changing the voltage from 2 300 volts to 110 volts for lighting purposes.

The large freight elevator (7 by 6½ ft.) serving all floors in the Administration Building, has a capacity of 10 000 lb. at a speed 75 ft. per min. The power is obtained from a 27-h.p. electric motor, operating at 900 rev.

per min. The machine shop is wired for machine tools, and a switchboard control panel has been installed.

In the foremen's room on the main floor of the Administration Building is located the wash-water Venturi meter register. The Venturi tube is 24 in. in diameter, with a 14-in. throat.

In this same room are two indicating depth gauges, one for each basin of the Baldwin Reservoir, and one pressure gauge to indicate the depth of water in the wash-water tanks. A high-water electric alarm bell notifies the operator when too high a level is reached in the wash-water tanks.

The chlorine room on the floor at Elevation 234.0 is located with one wall just beyond the outlet end of the filtered water conduit at Elevation 216.0. A pit, 1 ft. 6 in. by 3 ft. 6 in., and 2 ft. 3 in. deep, is built into the floor of this room. Passing through the wall of the pit are three 2½-in. galvanized-iron pipes, 7 ft. 6 in. long, and extending from 6 in. inside the pit wall to a point over the center line of the outlet end of the filtered water conduit. A manhole is located in the floor at Elevation 234.0, just over the end of these pipes. A heavy rubber hose fed by the chlorine machines may be placed in each pipe with its outlet end submerged in the water of the effluent channel leading to the clear-water reservoir.

For feeding the chlorine, there are two control units each with a maximum capacity of 320 lb. per day. The room in which these machines are located, is equipped with two vestibule entrances, each having two sets of double doors to prevent the spreading of fumes to other parts of the building. A ventilating system is also provided for exhausting the fumes from the chlorine room proper. The exhaust fan is located on the wash-water tank-room floor, and is driven by a 5-h.p. motor operating at 900 rev. per min.

An operating table is provided for each filter tank (Fig. 18). It consists of an ornamental cast bronze frame with statuary finish and with Tennessee marble panels and top. On the top are four levers equipped with indicators. These levers are used for opening and closing four-way cocks which admit and release water under pressure to the hydraulic cylinders used to open and shut the influent, effluent, sewer, and wash-water valves. A shut-off valve for the pressure manifold, and bronze casings for holding the loss of head and rate-of-flow gauge mechanisms, are also placed on the top of the table. In the hood of each gauge is a small electric bulb for illuminating the face of the gauge at night. The marble panels at the back of the table are omitted, the opening being closed, however, by a brass wire screen for ventilating purposes. The operating tables and fixtures were especially designed for this plant.

The 8-in. pressure supply line for operating the hydraulic valves is under a pressure of 150 lb. per sq. in. As this is too high for this purpose, it is reduced by an 8-in. pressure-regulating valve to about 110 lb. per sq. in. All waste lines from the four-way cocks discharge into the main gutter of the filter.

The effluent valve of each filter is placed down stream from the rate controller and this permits each controller to be emptied for inspection or repair without draining the effluent galleries. This arrangement, however, subjects the controller and connected mercury pots to the full head of the wash-water pressure. This necessitates the cutting off of wash-water pressures, during the washing process, from the mercury pots for the rate of flow and loss of head gauges. The effluent valve, therefore, when closing, automatically closes two small hydraulic valves in the small pressure pipe lines leading from the controller to the mercury pots. These devices prevent the mercury from being forced out of the pots while the controller is under pressure in the washing process. These cut-off valves open automatically when the effluent valve is again opened.

Lighting.—A very complete system of lighting has been installed in all the buildings. Special lighting for the filter tanks enables operators to view the filtering and washing processes at night. The offices and laboratories are extremely well lighted. Ornamental fixtures and lights are installed in the end and cross pavilions of the wings of the filter building.

Heating.—A low-pressure (5-lb. gauge and 10-in. vacuum) vacuum type of steam heating is provided for all the buildings except the gate-houses. Thermostatic control is used in the Administration Building. Steam is supplied from the boiler plant at the Fairmount Pumping Station.

Telephone System.—A 24-station intercommunicating telephone system furnishes contact between the offices, laboratories, foremen's rooms, and other important points in the plant. Duplicate sets of talking and ringing batteries are located in the machine shop, and are kept constantly charged by a motor generator set.

COMPLETION OF BALDWIN FILTRATION PLANT

The construction of the Baldwin Filtration Plant and Reservoir, as well as of all allied projects necessary to make the operation of this plant possible, was hastened in every way, in order to derive the benefit of increased pressures in the low-service district due to the use of the new reservoir.

All the various projects had been so nearly completed, that in June, 1925, it was possible to test and clean out the regular water channels in the Baldwin Filtration Plant from the chemical house through the filtered-water by-pass in the Administration Building, to Baldwin Reservoir. On June 29, 1925, raw chlorinated water was first passed through the previously mentioned water channels into Baldwin Reservoir and on July 1, 1925, this reservoir was connected to the low-service distribution system.

From the average results of twenty-two gauging stations, throughout the low-service district, the increased pressure due to the change to the higher reservoir was 16.6 lb. This was the difference between the average of the readings at 9:00 A. M., before cutting in the new reservoir, and that at 2:30 P. M., after the system had been changed over. This increase in pressure was obtained with the water in Baldwin Reservoir about 7½ ft. below the overflow.

The placing of the filter equipment progressed so satisfactorily that on September 15, 1925, the filtered-water by-pass was closed and the first chemically treated water was passed through the coagulation basins and to the west twenty filters, thus placing one-half the plant in service. On October 20, the remaining filters were available for service.

The total elapsed time required for the construction of the plant was slightly more than three years and eleven months. This is computed from the date of awarding the contract under which the reservoir was constructed, to the date on which all the filters were ready for service, and the plant was ready for output of its full capacity. It excludes the time required for the excavation of the reservoir and coagulation basins.

Amounts of Contracts and Analysis of Costs

The various items entering into the cost of Baldwin Filtration Plant are given in Tables 2 and 3. In preparing this detailed statement, the actual cost of the asset is given; this cost is also adjusted to a base cost in accordance with the purchasing value of the 1913 dollar. This was done in order to permit cost comparisons on the basis of pre-war values. The cost index figures as prepared by the *Engineering News-Record,* were used in making this adjustment. A comparison of unit costs may readily be made by means of Table 2. For example, the alum storage building, with a capacity of 1 000 tons, cost $132 per ton for storage. On the 1913 basis, the cost is $63 per ton of storage. Similarly, the four coagulation basins, with a total capacity of 32 800 000 gal., cost $49 425 per 1 000 000 gal. The capacity of Baldwin Reservoir is 135 000 000 gal., and its cost is seen to be $21 458 for each 1 000 000 gal. of storage.

Table 3 gives a summary of costs of the units comprising the filter plant proper, including both actual and 1913 values. It also gives for each of these values the cost of each unit per 1 000 000 gal. daily of capacity. This table excludes from consideration, the following items: (1) Mains delivering raw water to the rising well at the chemical house; (2) mains that take water away from outlet Gate-House No. 7; (3) changes at Fairmount Reservoir; (4) landscape development; (5) cost of all land; and (6) rough grading of the filter plant grounds, engineering, inspection, and overhead.

The large capacity of the Baldwin Reservoir is due to the fact that it is used as a storage reservoir for the low-service district. As the normal capacity of a filtered-water reservoir for a filtration plant is about 25% of the normal daily rated capacity of the plant, it is not proper to charge the entire cost of the Baldwin Reservoir to the filtration plant. Accordingly, while Table 3 includes costs for the plant as actually constructed, data are also included for the purpose of comparison, giving costs on the basis of the customary size of a filtered-water reservoir.

A capacity of 40 000 000 gal. would be sufficient for a plant of this size. The exact cost of this smaller reservoir could only be obtained by receiving bids based on an actual design, but for the purpose of comparison it will be assumed that the cost of the smaller, theoretical reservoir would be $\frac{40}{135}$ of the cost of the reservoir as actually built. The totals of Items A, B, C, D, and F, as given in Table 3, represent these adjusted values.

TABLE 2.—DETAILED COSTS OF BALDWIN FILTRATION PLANT.

Units constructed.	Actual cost of asset.	Cost reduced to 1913 basis by use of cost index figures.	Date of receipt of bids.	Cost index based on the year 1913 as 100.
Control Works at Fairmount Reservoir: Substructure:				
Contract 18, Foundations of four gate-houses, shafts, branch and main tunnels to header at pumping station, distributing pipe into reservoir, and all sluice-gates................	$ 209 891.76	$ 109 318.63	January 12, 1923	192
Contract 41, Guniting areas, concrete floor of reservoir, and slope wall paving around new openings made in floor and slope walls......	4 548.78	2 125.60	April 13, 1923	214
Total, substructure......................	$ 214 440.54	$ 111 444.23	....	...
Control Works at Fairmount Reservoir: Superstructure:				
Contract 42, Superstructures of four gate-houses..	$ 53 243.00	$ 24 201.36	October 24, 1923	220
Alum Storage Building (Foundation Area, 0.12 acres):				
Contract 57, Building complete, including foundations, superstructure, and alum-crushing and conveying equipment: Total....................................	$ 131 972.64	$ 62 844.11	January 2, 1925	210
Chemical House and Mixing Flume: Substructure and Equipment (Foundation Area, 0.65 acres):				
Contract 54-S, Substructure complete, including excavation and back-fill, concrete foundation and reinforced concrete skeleton of building, mixing flume, and chemical house equipment, including concrete dissolving, solution, and orifice tanks, piping and valves for chemical solution and alum-handling equipment........................	$ 378 407.54	$ 177 656.12	August 20, 1924	213
Contract 44, Additional work..................	493.50	227.42	May 25, 1923	217
Total, substructure and equipment.....	$ 378 901.04	$ 177 883.54	....	...
Chemical House and Mixing Flume: Superstructure:				
Contract 54-S, Superstructure complete, including all brick and cut-stone work, steel roof trusses, slate roof, partitions, sash, doors, painting, etc.........................	$ 215 000.00	$ 100 938.97	....	...
Elevator..	6 500.00	3 051.64	....	...
Electric wiring and lighting fixtures..........	16 500.00	7 746.48	....	...
Heating and ventilating........................	6 000.00	2 816.90	....	...
Plumbing complete.............................	5 000.00	2 347.42	....	...
Total, superstructure....................	$ 249 000.00	$ 116 901.41	August 20, 1924	213

TABLE 2.—(*Continued*).

Units constructed.	Actual cost of asset.	Cost reduced to 1913 basis by use of cost index figures.	Date of receipt of bids.	Cost index based on the year 1913 as 100.
Coagulation Basins: Substructure (Foundation Area, 7.74 acres):				
Contract 1, Excavation	$ 162 069.63	$ 61 158.35	April 20, 1920	265
" 2, Foundation under west conduit	33 521.82	20 193.87	November 4, 1921	166
" 44, Completion of excavation and constructing completely the concrete covered basins, including drains, proportionate cost of outlet end of main drain, foundations, walls, conduits, roof, roof back-fill, sluice-gates, mud-valves and pressure-pipe cleaning system	1 425 556.64	656 938.54	May 25, 1923	217
Total, substructure	$1 621 148.09	$ 738 290.76	....	...
Coagulation Basins: Superstructure:				
Contract 47, Superstructures of six gate-houses and brick and stone facing of west wall of basins	$ 106 000.00	$ 48 181.82	October 24, 1923	220
Filters and Administration Building: Substructure (Foundation Area, 2.48 acres):				
Contract 1, Excavation	$ 68 394.51	$ 25 809.25	April 20, 1920	265
" 44, Proportionate cost of outlet end of main drain	25 000.00	11 520.73	May 25, 1923	217
Contract 47, Completion of excavation, building drains, construction of all foundations, concrete effluent galleries, filter tanks, and conduits, placing all valves in pipe gallery and furnishing and placing all pipe gallery and effluent gallery piping	1 524 195.61	692 816.19	October 24, 1923	220
Contract 49-K, Furnishing hydraulically-operated filter valves	85 786.50	38 642.57	September 14, 1923	222
Contract 49-R, Furnishing hand-operated valves	15 109.40	6 806.04	September 14, 1923	222
Total, substructure	$1 718 486.02	$ 775 594.78	....	...
Filter and Administration Building: Superstructure:				
Contract 47, Superstructure complete above the tops of the filter tanks, including building skeleton and roof, all brick cut-stone work, metal lath, plaster, painting, etc	$1 250 530.00	$ 568 422.73	October 24, 1923	220
Contract 48-E, Electrical wiring and fixtures	31 000.00	14 090.91	October 24, 1923	220
" 48-S, Heating and ventilating system	34 659.00	15 754.09	October 24, 1923	220
" 48-W, All plumbing	29 775.05	13 534.11	October 24, 1923	220
" 54-B, Elevator	7 260.00	3 392.52	July 23, 1924	214
Total, superstructure	$1 353 224.05	$ 615 194.36	....	...

TABLE 2.—(*Continued*).

Units constructed.	Actual cost of asset.	Cost reduced to 1913 basis by use of cost index figures.	Date of receipt of bids.	Cost index based on the year 1913 as 100.
Filter Equipment:				
Contract 54-B, Furnishing and placing filter strainer system: furnishing and placing filter rate controllers and operating tables; furnishing and placing electrically driven wash-water pumps, transformers, and pump-motor control; furnishing and placing chlorine feeding apparatus; furnishing and placing office furniture, floor covering, and stock room shelving; and placing filter gravel and sand	$ 243 680.00	$ 118 869.15	July 23, 1924	214
Contract 52-C, Furnishing filter sand	33 550.00	16 286.41	November 6, 1924	206
Contract 52-O, Furnishing filter gravel	18 237.25	8 853.03	November 6, 1924	206
Contract 47, Steel wash-water tanks	13 500.00	6 186.36	October 24, 1923	220
Contract 47, Metal shelving, cupboards, and lockers	3 300.00	1 500.00	October 24, 1923	220
Laboratory work tables, cabinets, and benches	12 499.35	6 097.24	October 30, 1925	205
Total, filter equipment	$ 324 766.60	$ 152 742.19	...	...
Baldwin Reservoir: Substructure (Foundation Area, 13.81 acres):				
Soil stripping by direct labor in 1915	$ 26 386.00	$ 28 372.04	1915	93
Excavation contract	183 660.40	204 067.11	April 2, 1915	90
Excavation by direct labor	809 334.00	408 754.55	1919	198
Contract 2, Construction of concrete-covered reservoir complete, including completion of excavation, floor and wall drains, foundations, walls, roof, back-fill on roof, sluice-gates and all valves and pipe within Gate-House No. 7	1 873 735.36	1 128 756.24	November 4, 1921	166
Contract 47, Additional work	3 755 00	1 706.82	October 24, 1923	220
Total, substructure	$2 896 870.76	$1 771 656.76	...	...
Baldwin Reservoir: Superstructure:				
Contract 47, Superstructure of Gate-House No. 7 complete	$ 80 000.00	$ 36 363.64	October 24, 1923	220
Rough Grading of Filtration Plant Grounds: (Area of grounds, exclusive of area occupied by structures, 28.99 acres):				
Contract 1, Excavation	69 883.70	26 371.21	April 20, 1920	265
" 2, Excavation and fill	50 838.00	30 625.30	November 4, 1921	166
" 44, Fill	16 957.60	7 814.56	May 25, 1923	217
" 54-S, Fill	5 420.00	2 544.60	August 20, 1924	213
Total	$ 143 099.30	$ 67 355.67	...	...
Miscellaneous:				
Steam main to Plant A (July, 1925)	$ 25 200.00	$ 12 292.68	July, 1925	205
Steam main to Plant B (September, 1925)	9 720.00	4 811.88	September 25, 1925	202
Dump truck for hauling alum (October, 1925)	4 310.55	2 102.71	October 16, 1925	205

TABLE 3.—ACTUAL COST OF BALDWIN FILTRATION PLANT.

Item.	Units constructed.	Sub-structure.	Super-structure.	Filter equip-ment.	Total.
	SUMMARY OF TOTAL COSTS.				
A	Alum Storage Building	$ 131 972.64*	....	....	$ 131 972.64
B	Chemical House and Mixing Flume	378 901.04	$ 249 000.00	....	627 901.04
C	Coagulation Basins	1 621 148.09	106 000.00	....	1 727 148.09
D	Filters and Administration Buildings	1 718 486.02	1 353 224.05	$324 766.60	3 396 476.67
E	Baldwin Reservoir	2 896 870.76	80 000.00	....	2 976 870.76
	Total	$6 747 378.55	$1 788 224.05	$324 766.60	$8 860 369.20
F	Estimated cost of a 40 000 000-gal. reservoir $(=\frac{40}{135}\times \text{Item E})$	$858 332.08	$23 703.70	....	$882 035.78
	Total of Items A, B, C, D, and F	$4 708 839.87	$1 731 927.75	$324 766.60	$6 765 534.22
	TOTAL COSTS REDUCED TO COST PER MILLION GALLONS DAILY CAPACITY OF PLANT.				
A	Alum Storage Building	$ 799.83*	....	....	$ 799.83
B	Chemical House and Mixing Flume	2 296.37	$ 1 509.09	....	3 805.46
C	Coagulation Basins	9 825.14	642.42	....	10 467.56
D	Filters and Administration Buildings	10 415.07	8 201.36	$1 968.28	20 584.71
E	Baldwin Reservoir	17 556.79	484.85	....	18 041.64
	Total	$40 893.20	$10 837.72	$1 968.28	$53 699.20
F	Estimated cost of a 40 000 000-gal. reservoir $\left(=\frac{40}{135}\times \text{Item E}\right)$	$5 202.01	$143.66	....	$5 345.67
	Total of Items A, B, C, D, and F	$28 538.42	$10 496.53	$1 968.28	$41 003.23
	SUMMARY OF TOTALS ADJUSTED BY COST INDEX FIGURES TO CONFORM TO THE 1913 BASIS.				
A	Alum Storage Building	$ 62 844.11*	....	....	$ 62 844.11
B	Chemical House and Mixing Flume	177 883.54	$116 901.41	....	294 784.95
C	Coagulation Basins	738 290.76	48 181.82	....	786 472.58
D	Filters and Administration Buildings	775 594.78	615 194.36	$152 742.19	1 543 531.33
E	Baldwin Reservoir	1 771 656.76	36 363.64	....	1 808 020.40
	Total	$3 526 269.95	$816 641.23	$152 742.19	$4 495 653.37
F	Estimated cost of a 40 000 000-gal. reservoir $\left(=\frac{40}{135}\times \text{Item E}\right)$	$524 935.34	$10 774.41	....	$535 709.75
	Total of Items A, B, C, D, and F	$2 279 548.53	$791 052.00	$152 742.19	$3 223 342.72

* Includes superstructure.

TABLE 3.—(*Continued.*)

Item.	Units constructed.	Sub-structure.	Super-structure.	Filter equip-ment.	Total.
	TOTAL ADJUSTED COSTS REDUCED TO COST PER MILLION GALLONS DAILY CAPACITY OF PLANT.				
A	Alum Storage Building	$ 380.87*	....	....	$ 380.87
B	Chemical House and Mixing Flume	1 078.08	$ 708.49	....	1 786.57
C	Coagulation Basins	4 474.49	292.01	....	4 766.50
D	Filters and Administration Buildings	4 700.57	3 728.45	$925.71	9 354.73
E	Baldwin Reservoir	10 737.31	220.39	....	10 957.70
	Total	$21 371.32	$4 949.34	$925.71	$27 246 37
F	Estimated cost of a 40 000 000-gal. reservoir $\left(= \frac{40}{135} \times \text{Item E}\right)$	$3 181.43	$65.30	....	$3 246.73
	Total of Items A, B, C, D, and F	$13 815.44	$4 794.25	$925.71	$19 535.40

* Includes superstructure.

ACKNOWLEDGMENT

As this project has covered a considerable period of time, the work was carried on under four different City Administrations and great credit is due to the City's officials and Councils for their continued support in carrying the project to a successful completion.

The work has been conducted under the general supervision of the Directors of Public Utilities, Messrs. C. W. Stage, Thomas S. Farrell, A. B. Roberts, E. L. Myers, and Howell Wright, and Commissioners of Water, Messrs. C. F. Shultz, J. T. Martin, A. V. Ruggles, and C. P. Jaeger. During the entire development and construction of this project J. W. Ellms M. Am. Soc. C. E., was Engineer of Water Purification and Messrs. A. V. Ruggles and A. G. Levy, Members, Am. Soc. C. E., were Engineers of Construction and Survey. The Frazier-Sheal Company with J. E. A. Linders, M. Am. Soc. C. E., as Engineer of Design, and H. T. Hammer as Resident Engineer, had charge of the design and construction of Baldwin Reservoir. The design and construction of the remaining part of the project was handled directly by the Engineering Department of the Division of Water with A. G. Levy and G. W. Hamlin, Members Am. Soc. C. E., as Engineers of Design and the latter as Resident Engineer. Herman Kregelius, City Architect, designed all the superstructures in connection with this project.

The plant was completed and put into operation under the administration of City Manager W. R. Hopkins and Director of Public Utilities Howell Wright, to whose energy and ability the early completion of the project was largely due.

CONCLUSIONS

The project involved an expenditure of approximately $10 000 000, and contains engineering features which should be of interest to all those engaged in the design and construction of works of this type. The descriptions are in considerable detail in order to enable other engineers to grasp the essentials of the design and construction and to indicate the magnitude of the work. Certain features like the hydraulic-jump mixing flumes are a departure from the old methods of mixing chemical solutions with the water to be treated; the design of the coagulation basins in providing a straight flow through them with no baffles is an improvement which has proved entirely satisfactory in practice and which is a departure from existing methods; and the use of the sharply sloping floor in the coagulation basins for the dual purpose of reducing the height of the outside walls and facilitating the cleaning of the basins is of practical importance.

The design and construction of Baldwin Reservoir on account of its size, should be of considerable interest to engineers, since it is the largest covered concrete reservoir that was ever constructed. The methods of introducing the water to the reservoir and drawing it off in order to provide good circulation is a point of interest. The groined arches used for the roof have the longest span constructed to date.

The very complete tables of costs are so rarely found in engineering literature that the figures given should be particularly valuable since they represent the careful analysis of all the contracts involved in the project.

The plant is adjacent to the Park System of the city and a proper architectural treatment was essential. The buildings present a most imposing appearance and the beauty of the design as a whole has been rarely, if ever, equalled.

DISCUSSION

S. M. VAN LOAN,[9] M. AM. SOC. C. E. (by letter).—This paper has been written in a most interesting manner and the description of the plant, together with the tabulated costs, is of high value to all construction engineers.

The writer has been impressed by the structural beauty of the exterior and interior of the Baldwin Filtration Plant. During a visit in 1926, his interest centered upon the influent to the coagulating basins. At the time, the hydraulic-jump mixing flume was under operation. It would be interesting to know whether this installation has given the results which were hoped for and which were expected, and if it bears out in actual operation the expectations that were forecast by the experimental flume.

In June, 1926, the filter beds were operating under a burden of organic growth, and numerous cyclops were noticed on the sand where water was below the surface. Has much trouble been experienced in the operation of the beds due to the organic growth in the applied water? Have taste-producing substances been injected into the lake water so as to contribute either a taste or an odor to the influent of the Filtration Plant, and, if so, what chemical applications have been made for their reduction or elimination? Information has reached the writer indirectly that experiments with ammonia have been made in Cleveland, and it would be of interest to know how far this research work has been carried and what assumptions have been noted.

A reply to these operating questions will be of great value to the filter plant operator regardless of the type of plant on which he may be engaged.

HARRY N. JENKS,[10] M. AM. SOC. C. E. (by letter).—The authors have placed the Sanitary Engineering Profession greatly in their debt for the wealth of technical data and interpretative comment which they have furnished. In discussing a paper of such scope, one is naturally inclined to limit one's self to a consideration of certain specific topics, because only those intimately acquainted with the design conditions and requirements are in a position adequately to evaluate the merits of the conception of the filtration project as a whole.

Following the order of presentation found in the paper, it is significant to note the prominence given the hydraulic data and the influence that hydraulic values have on the satisfactory operation of the plant itself. In particular, the proper control of velocities of flow throughout the plant is considered of great importance to the success of each stage of the process.

One of the most important elements in successful coagulation is the provision of adequate mixing velocities, and the subsequent preservation of the floc, so essential to satisfactory sedimentation.[11] A distinguishing feature of the mixing stage in the Baldwin Filtration Plant is the hydraulic-jump flume installation. The value of this initial rapid mixing is rightly becoming more generally recognized. The writer has obtained excellent results in coagulation by passing the water through an aeration system with a small dose of alum to produce a pin-point floc. Thorough coagulation ensued rapidly thereafter when the remainder of the total dose of alum was added. Under these circumstances, also, the aerators served the useful purpose of dissipating the quota of carbon dioxide resulting from the application of the initial dose of coagulant.

---

[9] Deputy Chf., Bureau of Water, Philadelphia, Pa.

[10] Cons. San. Engr., Berkeley, Calif.

[11] "Elements of Successful Coagulation and Filtration," *Water Works*, Vol. LXVII, No. 5 (1928).

From Table 1 it is noted that the observed loss of head through the flume is 2.40 ft. This is in favorable comparison with that encountered in baffled basins or channels for an equal degree of mixing. From the basic data submitted, it is seen that the velocity of the water in the flume is 12.5 ft. per sec. The initial mixing period is, therefore, at most only a matter of a few seconds. The question one might raise is whether full returns for the loss of head of 2.40 ft. are obtained under this time limitation. It seems that a more commensurate initial jarring of the water might be obtained through mechanical mixing. The flume, however, is a very compact device and one that requires no operating attention. Would it not also be possible to construct it so as to make it a means of measuring the flow, in the manner of the Parshall flume?

The use of mechanical mixing devices is deservedly gaining favor among designing engineers and operators of water treatment plants. It is generally desirable to provide for decreasing mixing velocities, which may be obtained through the use of stirring mechanisms operated at corresponding variable speeds. In one of his installations the writer introduced an original design which he has termed "spirovortex" mixing, wherein a controllable mixing velocity is maintained in the coagulation tanks by means of the recirculation of a part of the flow by pumping. Results of experiments indicate the desirability in some cases of providing for chemical agitation as distinguished from mixing as such, to which the "stream-flow" method of recirculation over a riffled surface is particularly well adapted. This in effect involves a series of small hydraulic jumps.

Intermediate between the well-ordered mixing of coagulation and the semi-quiescence of sedimentation, a period of very gentle stirring or streaming should be interposed to produce agglomeration of the floc. This takes place at velocities ranging, for typical waters, from 0.1 to 0.15 ft. per sec. The tank or channel capacity required for this period is more than offset by the resulting possible decrease in the size of the sedimentation basin.

It is significant to note the authors' care in providing for sufficiently low velocities in the coagulated water channels to prevent the destruction of the floc. It is true that an endeavor should be made to keep the velocities within the range of 1.5 to 2.0 ft. per sec. The writer is inclined to the belief, however, that it is not so much the specific velocity as the character of the flow itself, which is the governing factor. It seems that the floc may be unharmed even if considerably higher velocities are present, provided the flow is smooth and free from excessive eddies. Abrupt changes in direction, causing impact, represent the most unfavorable conditions. Furthermore, the entrainment of air, or the agitation of the water by air, is conducive to the destruction of the large flocs, and their re-solution into a colloidal state. In general, channels and gate openings which have good hydraulic conditions involving low hydraulic losses are favorable to the preservation of the floc, especially if the hydraulic radius of the channel is large, thus inducing a relatively small amount of "drag" on the side walls in comparison with the volume of flow.

From Table 1 one may note the close agreement between the computed and observed loss of head through the pre-treatment portion of the Baldwin Filtration Plant. Such information should be of value to designers in making an appropriate allowance for such loss. In this connection Fig. 19 will be of interest. This is a balanced curve and is drawn through the plotted points of observed losses of head. The nominal design of capacity of the plant to which this curve refers, was 32 000 000 gal. per day, a rate at which the total loss of head is seen to be 1.8 ft., increasing to 5.0 ft. at the designed maximum flow of 48 000 000 gal. per day.

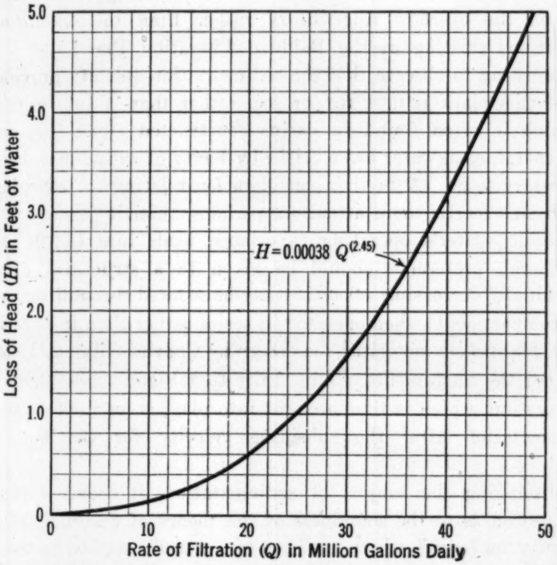


FIG. 19.—LOSS OF HEAD THROUGH SACRAMENTO PRE-TREATMENT WORKS, BETWEEN COAGULATION TANK NO. 1 AND THE FILTERS.

During recent years it has become apparent to sewage works engineers that optimum conditions for sedimentation are more dependent on basin area than on crude detention period. Advantage is now being taken of the important influence of the area factor in the design of relatively economical, shallow basins for water filtration plants, the basins being provided with continuous sludge removal. Current investigations by the writer indicate that hydraulic means for the satisfactory removal of the sludge may be applied on a large scale.

The authors' data show that the following significant factors entered into the design of the Cleveland coagulation basins:

Total net settling area (inside of basins), in square feet..	283 200
Average depth of water, in feet........................	15
Velocity of flow, in feet per second (at 2.36 ft. per min.)..	0.04
Detention period, in hours............................	4.7

Based on a flow of 165 000 000 gal. per day, or 114 500 gal. per min., the area allowed for sedimentation equals 2.47 sq. ft. per gal. per min., which is a high enough value for this factor to account for the excellent sedimentation efficiency found at the Baldwin Plant. Since the volume of flow is 917 000 cu. ft. per hour, the conditions of operation permit an overflow rate of 3.25 ft. per hour. This appears to be quite conservative for alum floc.

The 15-ft. depth of water is a large factor in the provision of the 4.7-hour storage period. It would appear possible in many cases to effect a more economical design through fixing the depth factor at, say, 8 ft., and installing means for continuous sludge removal. It would be of value to learn from the authors their opinion as to the relative merits of coagulation basin design predicated on the overflow rate theory rather than the detention period, under conditions existing at the Baldwin Filtration Plant.

It is interesting to observe that the sedimentation velocity provided by the designers of this plant is 0.04 ft. per sec. rather than a substantially lower velocity which is often found in basins constructed along less economical lines. In general, each water has a critical velocity of subsidence, below which it is unnecessary to go. A further consideration relating to velocities of sedimentation basins merits more attention, perhaps, than it has received among designers. The writer's operating experience leads him to the conclusion that, in general, preference should be given to a structural arrangement whereby gradually decreasing velocities may be secured through the sedimentation basin compartments when operated in series rather than in parallel. Such a functional feature characterized the design by Charles Gilman Hyde, M. Am. Soc. C. E., for the Sacramento, Calif., Filtration Plant. The phenomenon of floc agglomeration, which is thus brought into play, is an important factor in rapid sedimentation after the subsiding velocity for the floc is finally attained.

While discussing the subject of sedimentation, it seems appropriate to raise the question as to the soundness of the theory of design implied by the term, "coagulation basin", which has been so generally applied to the structure intended for the settlement of the precipitated matter from the water passing through the basins. It is a matter of considerable surprise to observe the tendency that has prevailed in filtration plant design, involving inadequate chemical mixing and a dependence on the sedimentation basin for the actual completion of the required reactions of coagulation. In the interests of economical design, as well as of economy and flexibility of operation, the stages of coagulation and sedimentation should be complete in themselves, and distinctly separate from each other. When means are provided for thorough coagulation (preferably with agglomeration), it will generally be found that the size of the sedimentation basins as such may be reduced to a degree that will more than compensate the cost of prolongation of the chemical mixing period. Then, with proper inlet and outlet arrangements and with long, narrow basins, similar to those at the Baldwin Plant, it is possible to obtain exceptionally high sedimentation efficiency, as measured by the removal of suspended

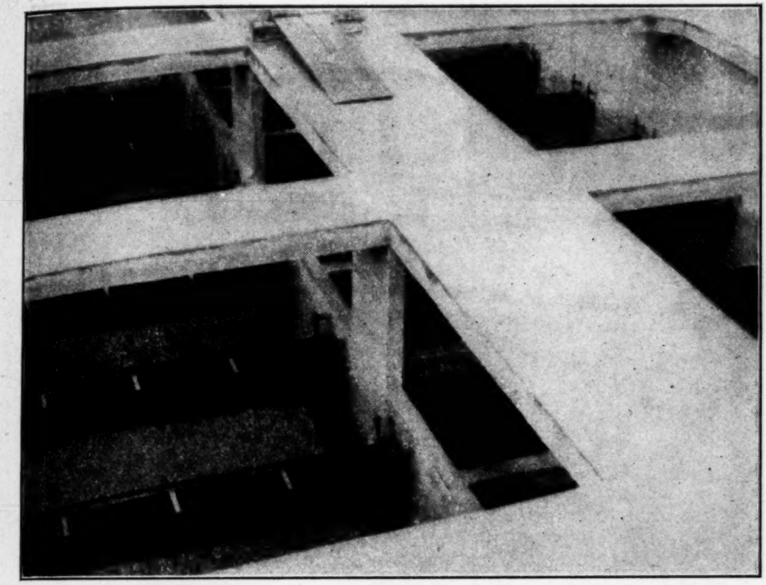


FIG. 20.—VIEW OF WASH-WATER GUTTERS IN SACRAMENTO FILTRATION PLANT, ADJUSTABLE IN ELEVATION BY MEANS OF HANGER BOLTS AT EACH END.

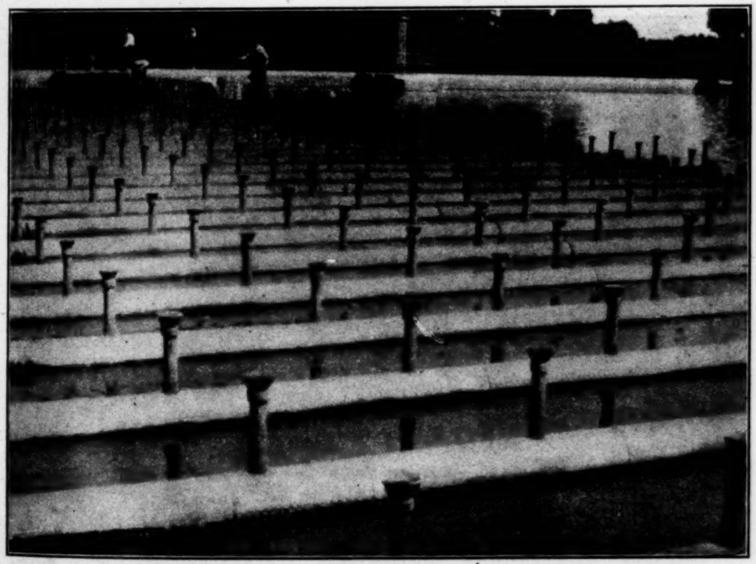


FIG. 21.—AERATION LATERAL AND RISER SYSTEM, DESIGNED ON BASIS OF HYDRAULIC PRINCIPLES INVOLVED IN PROPORTIONING THE ELEMENTS OF PERFORATED-PIPE FILTER UNDERDRAINS, SACRAMENTO FILTRATION PLANT.

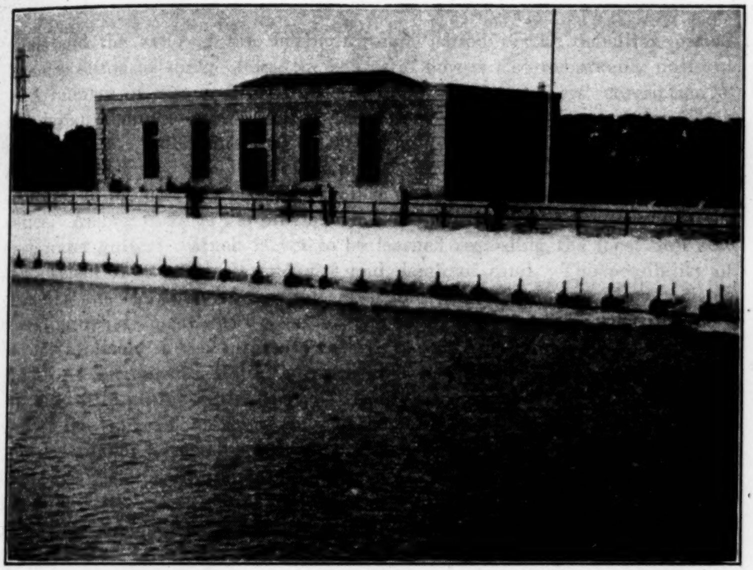


FIG. 22.—SACRAMENTO AERATION FIELD IN OPERATION, SHOWING UNIFORMITY OF DISCHARGE OVER ENTIRE AREA, THROUGH FLOATING CONE NOZZLES MOUNTED ON PIPING SYSTEM OF FIG. 21.

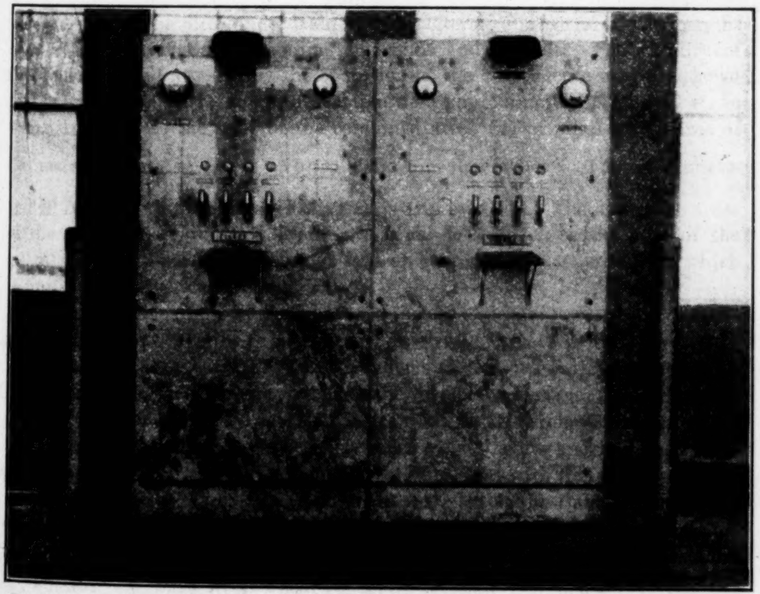


FIG. 23.—ORIGINAL ELECTRICALLY OPERATED FILTER CONTROL PANEL BOARD INSTALLATION AT SACRAMENTO, CALIF.

solids and the ratio of the flowing-through period to the detention period. The deposition of sludge along the course of flow is also remarkably uniform.

A number of interesting points are raised in the authors' description of the design features pertaining to the filters. The capacity of each filter, amounting to a little more than 4 000 000 gal. per day, is in keeping with the modern tendency toward large-sized units. The design of the filters in detail conforms to the best engineering practice, and several features deserve special notice. Among these is the provision for the adjustment of the height of the wash-water gutters. Much is yet to be learned regarding the most desirable and efficient rates of wash, and size and depth of sand. The possibility of changing the elevation of the gutters enables the operator to meet variable operating requirements, as well as to take advantage of information that may become available in the future as regards the physical functions of the filter itself. At Sacramento, Calif., adjustment is made by bodily raising or lowering the entire gutter, which may be readily accomplished through manipulation of the hanger-bolts shown in Fig. 20. At Fort Dodge, Iowa, the writer provided similar adjustable gutters, primarily with a view to enabling appropriate modifications to be made whenever a change from deep wells to the Des Moines River as a source of supply should become necessary.

The authors describe a strainer system designed in accordance with results of experiments by Mr. Ellms. There are two typical situations of this kind in a filter plant, (1) the wash-water manifold and laterals in a filter; and (2) the header and pipe system for an aeration field. The latter is illustrated in Fig. 21. One of the authors has shown[12] that the hydraulic theory applicable to the design of the filter underdrain system (laterals and perforations) may be used successfully in proportioning manifolds and laterals. The validity of this theory is well attested by the operation of the Sacramento aerators shown in Fig. 22. This view shows an aeration field approximately 172 by 40 ft. in plan. In this case the ratio of the combined areas (a) of pipes to the area of the main header (A) is, $\frac{a}{A} = 0.41$. The pipe laterals are 10 in. in diameter and 40 ft. long, while the header channel is 12.5 by 4.5 ft. The aggregate areas of the nozzle risers in each lateral are equal to four-tenths the area of the lateral. There are 420 sprays spaced 4 ft. on centers, the discharge from which is uniform at all rates (see Fig. 22).

It is significant of the tendency toward the use of deeper gravel beds that the authors specified 22 in. of graded gravel in the Baldwin Filters. The writer is also in agreement with the use of smaller sizes of rock than has often been furnished. His own experience confirms the wisdom of having a relatively large portion of the total depth comprise rock ranging in size from ¾ to $\frac{1}{16}$ in. This adds materially to the stability of the upper gravel layers and is a good safeguard against the intermingling of the overlying sand with the gravel beneath.

In connection with the filter equipment installed at the Baldwin Plant, the provision of only two gauges for each filter, namely, the wash-water and loss-of-head gauges, is in reality all that is required. From the standpoint of economy durability, and attractiveness, one may regard with favor the development of filter operating tables in the form of electrically-operated panel boards. The first installation of this type was made under the writer's direction at Sacramento,[13] and, subsequently, it was introduced in the water-works design at Fort Dodge, Iowa. Fig. 23 shows the original Sacramento panel board. The provision of electrical indicating instruments obviates the requirement of a specific position of the control panel to fit the location of float-tubes and accessories; also, all actuating strings and rods can be eliminated, particularly on the valve-opening indicators, which are readily actuated by reactance coils responding to the movement of the valve stem followers.

---

[12] *Journal*, Am. Water Works Assoc., Vol. XVIII, December, 1927, p. 664.

JAMES H. HERRON,[14] M. AM. SOC. C. E. (by letter).—In this paper the excellent compilation of cost data is one of the most valuable features. While the improvement was made during the period of high costs, the proper index can be applied to make them available for any period.

It has been a source of gratification that the plant as finally planned has worked out so well, since both the Cleveland Water Department and the Commission of Engineers, of which the writer was a member, were severely criticized for separating the Filtration Plant from the initial Pumping Station. Difficulty had been experienced with sub-grade conditions at one of the other plants located on the shore of the lake, and it was deemed impossible without high cost to build this plant adjacent to the Pumping Station. Since the Baldwin Reservoir had already been planned as a distributing reservoir for the low service, it was thought wise to avoid the expense of an additional filtered-water reservoir by utilizing one already available. The cost of pumpage is little different whether the water is delivered to the Filtration Plant through long lines and into the reservoir through short lines, or *vice versa.* The separation of the plants may have involved some additional pumping capacity, but with little greater total head. The increased investment was more than offset by the saving effected in utilizing a reservoir already planned and practically built as a clear-water basin.

In the building of the plant the utmost care was taken in the selection and handling of materials. The aggregates especially were approved only after examining all available supplies.

In the concreting operations great care was used in the grading of materials and in the quantity of water. This latter was kept at a minimum. Between 300 and 400 tests were made on concrete cubes (tests on cubes were then required by the Building Department of the City of Cleveland), with the following average results:

Nominal mix.	Strength at 28 days, in pounds per square inch.
1:3:5	2 475
1:2:4	3 100
1:2.5:3.5	3 400
1:3:3	3 875

---

[13] "Electrically-Operated Gages Devised for Filters," *Engineering News-Record,* Vol. 95, No. 9 (1925).

[14] Cons. Engr., Pres., The James H. Herron Co., Cleveland, Ohio.

The higher strength mixes were used for the important work, and the results fully justified the care exercised.

The concreting operations, both as far as the proportions of the materials and placement were concerned, were in advance of the general state of the art then prevailing. As a result the structure has shown no signs of disintegration or other deterioration, and should give continued satisfaction.

E. SHERMAN CHASE,[15] M. AM. SOC. C. E. (by letter).—The writer has been impressed not only by the size of the Baldwin Filtration Plant, but by the general excellence of its design and construction. It is gratifying to have so complete a description of the plant available in permanent form.

The use of the hydraulic jump for bringing about the mixing of the coagulating chemicals with the water is one of the interesting features of the plant. This method has the obvious advantage of simplicity and absence of mechanical equipment. On the other hand, there is a loss of head involved of approximately 2½ ft., which could have been largely avoided had mixing tanks equipped with stirrers been installed. Furthermore, stirring devices capable of being operated at variable speeds permit somewhat greater flexibility of operation than flumes of fixed dimensions.

The application of the aluminum sulfate in solution rather than in the dry form is a departure from what seems to be the present trend in filter plant practice. It is probable, however, that in a large plant with an adequate laboratory staff, the adjustment of the alum dose can be regulated more exactly by the method adopted than by dry feed. Economy in the use of coagulants is of substantial merit where large quantities of water are treated daily.

The adjustable weir plates on the wash-water troughs are novel features and the wisdom of this provision has apparently been substantiated in practice as shown by the experiences described by W. C. Lawrence, Superintendent of Filtration at the Baldwin Plant.[16] According to Mr. Lawrence difficulties in securing proper cleaning of the sand grains during filter washing were overcome by increasing the rates of vertical rise with increased temperatures of the raw water. Had fixed overflow weirs been provided at the lowest elevation available, it would have been difficult to have readily increased the rate of wash without undue loss of sand.

The advantage of high-velocity wash appears to have been shown elsewhere. This is particularly true at the filter plant at Greenwich, Conn., and has been demonstrated experimentally at Detroit, Mich.[17] It seems probable that in the future more liberal provision for high wash-water rates will be made in filter-plant design than has been the general practice in the past.

The characteristics of the filter sand closely follow the usual specifications. Experience in the Greenwich plant indicates that the requirements for a low uniformity coefficient may be unnecessary in the case of rapid sand filters.

---

[15] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

[16] "Variable Wash Water Rates with Changes of Water Temperatures," *Journal*, Am. Water Works Assoc., Vol. 22, No. 2 (1930).

[17] "Studies on the Washing of Rapid Filters," by Messrs. Hulburt and Herring, *Journal*, Am. Water Works Assoc., Vol. 21, No. 11 (1929).

At this plant, six of eight filters were provided with sand having an effective size of 0.36 to 0.37 mm. and a uniformity coefficient of 1.48 to 1.55, whereas, in two other filters, the sand had an effective size of 0.37 mm. and a uniformity coefficient of 4.65.[18] Operation records have demonstrated equally satisfactory results with the two sands. The reason for equal performance is probably due to the fact that the hydraulic grading of the sand by washing results in the finer sand being brought to the surface of the bed where most of the purification is accomplished.

In closing, it is interesting to note that the City of Cleveland has expended about $7 000 000 for the purification of a supply which, as late as 1910, was considered fairly satisfactory even without chlorination. This fact illustrates the increasing demand on the part of the public for water supplies of the highest degree of purity.

J. W. Ellms, G. W. Hamlin, A. G. Levy, and J. E. A. Linders, Members, Am. Soc. C. E. (by letter).—The engineers who have so kindly discussed this paper have brought out a number of points in the design which are worthy of further comment.

The question asked by Mr. Van Loan relative to the operating results with the hydraulic-jump mixing flumes, may be answered by calling attention to the results obtained from five years of successful operation. These flumes have efficiently mixed the alum solution with the raw water, and the flocculation following the jumps has been entirely satisfactory.

The use of an alum solution instead of dry feed does depart from the general trend of present-day practice, as mentioned by Mr. Chase. The writers are fully aware that this is the case, but experience has shown that accurate and uniform, and consequently economical, application of the coagulant has been obtained at the Baldwin Plant.

The preparation of a dilute alum solution of uniform strength by the method used at the Baldwin Plant, namely, the dissolving of a weighed quantity of alum by the upward flow of a measured volume of water, is an extremely simple process; and when the solution is properly agitated before using, does not require laboratory supervision. The writers are firmly convinced that the continuous uniform measurement of a liquid is more accurately and more easily obtained than the continuous measurement of solid particles.

The experimental work[19] upon which the design of the hydraulic-jump flumes was based, conclusively demonstrated the thorough mixing action produced by this method. In this work the exploration of the jump with Pitot tubes showed water currents moving in all directions within the jump. The writers cannot conceive of any form of mechanical stirring which would produce "a more commensurate initial jarring of the water", as stated by Mr. Jenks, than that obtained in the hydraulic jump.

The Baldwin Plant was designed, in the section providing for the preliminary treatment, on the basis of two steps: First, for a practically instantaneous, thorough, and intimate mixing of the chemical coagulating solution with the water; and, second, for a gentle agitation of the entire flow of the treated water over a period of time to permit of the agglomeration of the floc. The purpose of the first step was simply one of mechanically mixing the alum solution with the water, and the element of time is immaterial provided thorough mixing is effected. The elements of time and velocity, however, do enter into and play a very important part in the second step of the process, namely, the agglomeration of the colloidal particles into a floc formation. The second step in the process is accomplished at the Baldwin Plant in the large conduit, equipped with sets of "over and under" baffles, which connects the hydraulic-jump mixing section with the coagulation basins. The length of time required for this secondary agitation depends upon the character of the water being treated, and the quantity of coagulant added. In experimenting with Lake Erie water, the writers found that this time period need not be longer than 5 or 6 min., with a velocity of 0.75 to 1.00 ft. per sec. A pin-point floc is characteristic of Lake Erie water treated with alum as the coagulant. The usual small quantity of suspended matter in Lake Erie water may account for the small size of the floc formed.

---

[18] "The New Purification Plant of the Greenwich Water Company," *Journal*, New England Water Works Assoc., Vol. LXII, No. 4 (1928).

[19] "The Hydraulic Jump as a Mixing Device," by A. G. Levy and J. W. Ellms, Members, Am. Soc. C. E., *Journal*, Am. Water Works Assoc., January, 1927.

The writers agree with Mr. Jenks that the loss of head through the hydraulic jump is comparable with that in the conventional type of baffled mixing chamber; but they seriously question whether the same degree of thorough and rapid mixing of the chemical with the water is obtained in the baffled chamber.

The design of the Baldwin coagulation basins was based on a theoretical velocity of flow and a definite period of detention. It was considered desirable to have an average velocity of 2 to 2.5 ft. per min., and a detention period of 4.5 to 5 hours.

Continuous sludge removal was not permissible at the Baldwin Plant because the sludge could only be removed in the spring and fall months.

More shallow sedimentation basin design than is commonly used appears to have a reasonable basis from theoretical considerations. However, certain practical deductions from observations extending over many years, with many different types of water, lead the writers to believe that efficient sedimentation of suspended matter, with a specific gravity as close to that of water as is produced by coagulating Lake Erie water with alum, requires average depths of not much less than 15 ft. It will have to be proved that in so mobile a liquid as water the velocities in basins of only about one-half this depth are not destructive of efficient sedimentation in the lower strata of water at and near the bottom. The writers believe caution should be used in applying the over-flow rate theory as at present developed. One questionable point is the relationship between total depth and the overflow rate depth. Whatever factor may be used for this ratio would seem to be affected materially by the character of the suspended particles. As variations in temperature of the water will change its viscosity, it does not seem improbable that efficiencies of coagulation basins may be affected by this fact, as noted by Mr. George G. Nasmith.[20]

---

[20] "The Trial Filtration Plant, Ottawa, Canada," by George G. Nasmith, *Journal*, Am. Water Works Assoc., August, 1930.

The hydraulics involved in the design of a filter plant is of the utmost importance and affects its successful operation. Experience has proved that the greatest care and skill are required in making the computations involved in the various hydraulic problems entering into the proper design of a filtration plant. When these computations are carefully made, it will be found that the actual hydraulic gradient through the filtration plant will be in very close agreement with the theoretical gradient as calculated.

Good design involves the proper velocities in mixing the coagulant with the water, in secondary agitation to produce proper flocculation, in channels and through sluice-gates to prevent breaking up the floc, and in coagulation basins to permit of effective sedimentation. Adequate available head for filter operation and for washing filters should also be provided. Gutter and drain capacities for wash water, when used at the highest rates of flow, are imperative to permit proper cleaning of the filters. Unless all these points of design are carefully considered and properly calculated an efficient plant cannot be constructed.

An additional feature of the purification process at the Baldwin Plant not originally contemplated has been the adoption of the ammonia-chlorine process of disinfection, by which the problem of chloro-phenolic tastes appears to have been solved. No such objectionable tastes have been noted since the adoption of this process at the Cleveland filtration plants in January, 1930.

The writers are glad to note that Mr. Herron finds the cost data to be of value and in convenient form for use. The writers believe them to be one of the most important features of the paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

PAPERS AND DISCUSSIONS

Paper No. 1774

LAMINATED ARCH DAMS WITH FORKED ABUTMENTS[1]

By Fred A. Noetzli,[1a] M. Am. Soc. C. E.

With Discussion by Messrs. Lars R. Jorgensen, William Cain, B. F. Jakobsen, P. Wilhelm Werner, H. B. Muckleston, Fredrik Vogt, Paul Bauman, F. W. Hanna, A. A. Eremin, and Fred A. Noetzli.

Synopsis

This paper describes some modifications of arch dam form and layout, as compared with usual current practice, which are believed to promote both simplicity of design and economy. Simplicity of design is achieved by the use of structural proportions that avoid the complexities of the "thick arch", and economy, fortunately, follows in consequence.

That the controlling principles hereafter noted, and the ends to be sought, are now recognized in some quarters, is evidenced by several recent dam designs, but it is believed that this conception may advantageously be carried further than it was in these instances. The purpose of this paper is to present the orderly development and application of certain basic principles, carried to their logical conclusion.

It is apparent that the efficient concrete arch is one in which the stresses are approximately equal across any section; that is, in which the line of thrust approximately coincides with the neutral axis. This condition is most nearly obtainable in a dam composed of thin arches; it is impossible to secure in "thick arches." Furthermore, as arches depart from the "thin" classification, and approach the "thick", the relation between the radius, the thickness, and the central angle, has important consequences upon the stress distribution; and, other things being equal, the maximum angle will generally give

---

[1] Published in February, 1930, *Proceedings*.

[1a] Cons. Hydr. Engr., Los Angeles, Calif.

minimum stress. With these considerations in view, a typical structural form and layout has been developed by the writer, such as will permit their attainment to the maximum practical degree. The topography at the dam site is obviously a controlling feature but, for typical conditions, the desired results are obtained by:

1.—The use of a special type of buttress to take the thrust of the upper arch elements, thereby reducing the required maximum arch span, and hence the maximum radius.

2.—The pronounced sloping, up stream from the crest, of the crown of the dam, thereby securing the minimum radii for the lower arch elements.

3.—By the subdivision, where indicated, of the total arch thickness into two or more separate arches, or arch laminæ, thereby keeping the arches "thin", or relatively so, and maintaining throughout the dam the corresponding advantages.

Principle 1 requires the use of gravity wings which are so related to the buttresses as greatly to increase the strength of the latter, and to provide ample assurance against over-stress in this quarter. Principles 1 and 2 have been applied by the writer to the design and construction of a dam hereafter described. The method of lamination has been adopted in principle by French engineers on a large dam. It is believed that the method proposed by the writer is equally as effective and possesses some advantages over the French design.

INTRODUCTION

The first arch dams were designed by the simple cylinder formula. Most of these structures are of moderate span and height, and the unit stresses assumed in the design were relatively low. No arch dam designed by this method has failed for reasons of structural weakness. This speaks rather well for the adaptability of such structures to the loads and stresses caused by the water pressures.

A great improvement in the design of arch dams accompanied the application of the elastic theory. Few other branches of structural engineering have received, during the years since 1920, as much attention as dams of the arch and multiple-arch type. This is evidenced by the large number of theoretical studies published during this period in the United States and in foreign countries.[2] Tests and experiments were made on a number of dams and on laboratory models, to determine within reasonably close limits the laws of the distribution of stresses in arch dams. The most signal advance in this connection was the construction and successful testing of the Stevenson Creek Experimental Arch Dam by the Committee of Engineering Foundation on Arch Dam Investigation. Much valuable and useful information has been obtained from these theoretical and experimental studies.

---

[2] See Bibliography in Report on Arch Dam Investigation, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 277.

Theoretical investigations have shown that, for thick arches of dams, certain assumptions ordinarily made in the analysis of relatively thin arches—as used in arch bridges and arch dams of moderate height and span—could be improved upon, and the theory of thick arches was developed. However, apparently it was found that this theory has a rather limited field of application, since it often becomes necessary to abandon the actual arch as a structural element, and have recourse to a contained "secondary arch".[3]

This "secondary arch" is assumed to support the entire water load, and is analyzed accordingly. The basic assumption related to these secondary arches, inside the primary thick arch, is that numerous cracks are formed in the tension zone of the latter; that is, on the extrados at the abutments and on the intrados at the crown. In other words, the most advanced theory serves to show that, in some instances, the structure cannot function as built, and in such cases it is customary to neglect the concrete that is presumably cracked or in tension. The conditions which determine the secondary arches are very complex, and involve some rather uncertain assumptions; but even if this solution were clean cut and definite, it would still be open to the objection that to place deliberately in a dam concrete which it is assumed will be structurally inoperative, is not in accord with efficient design or engineering.

In his early studies of arch dams the writer was impressed with the difficulties involved in a theoretically exact analysis. His further studies, and the knowledge derived from the published work of others, have served to strengthen his original impression. It appears that each advance made in the development of the structural theory has not only introduced additional mathematical complications, but necessarily has involved fundamental assumptions which are admittedly uncertain of determination. In other words, despite the great advance that has been made in mathematical analysis of arched dams in recent years, the inherent complexities and uncertainties of the problem are still present.

When a structural type is such that its analysis is complicated and uncertain beyond a reasonable degree, and when it is evident that primary structural elements cannot function as built, a modification of type, such as to result in simple relationships, is perhaps indicated. Great complexity of design can only be justified by producing corresponding efficiency, which is questionable in the case of the present thick arch dams.

Some of the major difficulties encountered in the analysis of the current type of arched dam are results of the use of thick and flat arches. The consequent complications introduced are twofold: First, as regards the action of the arches themselves, and, second, the action of the stiff cantilever elements, and their influence upon the arches. It is known that the action of a thin arch under water loads, as in the upper part of a thin arched dam, is relatively simple and determinate. It may also be readily shown that, regarding arching action only, the thin arch is usually far more efficient than the thick. So far as the arches themselves are concerned then, it would obviously be desirable to keep them as thin as possible, within practical limits. As the thickness of the arches is reduced, the flexibility of the vertical or cantilever elements is increased in very much greater proportion, resulting in a greatly reduced influence of these elements, and an increase in the proportion of load carried by the arches.

---

[3] *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 475.

On the other hand, an increase in the efficiency of the cantilever elements (by increasing their thickness) can only be accomplished at the expense of reduction of arch efficiency. In other words, it is assumed that the thin and efficient arch may be taken as the goal of attainment, without any material sacrifice of other benefits.

With the preceding considerations in view, the writer for some time past has devoted his efforts toward the development of a type of arch dam which would eliminate or reduce some of the difficulties and complexities of the current thick arch type, and, at the same time, give a structure of greater efficiency. The purpose of this paper is to describe the type of structure evolved and the manner in which the desired ends are accomplished.

In what has preceded, it was stated that the thin arch, subjected to water loads as in a dam, possessed marked advantages over the thick arch under similar conditions. This fact is probably well understood, but it will be pertinent to summarize here these relations in greater detail.

As has been shown by recent investigations, the influence of rib-shortening, temperature, and shrinkage becomes very marked as the arch thickness is increased. Their effect is to create an eccentricity of the resultant thrust, and, therefore, an unequal distribution of pressure across the section, tending toward or developing tension at the crown intrados and at the extrados at the abutments. The fact that these important influences are neglected in the cylinder formula is one of the main reasons that this is not applicable to thick arches.

In all, or most all, the theoretical investigations of the arched dam, the analysis has been based upon horizontal elementary arches, consequently subjected to uniform normal water pressure. That is, these are assumed to be the primary elements, or those carrying the maximum stress. Depending upon the layout of the dam, and other considerations, the actual primary elements may be horizontal or inclined, but it is probable that conclusions pertaining to horizontal elements will be applicable in kind to inclined elements, and will serve as a measure of the conditions obtaining in the latter. The complexities introduced by a consideration of inclined elements would prohibit any generalized theory or conclusions, in the interests of which it is permissible to adopt the horizontal arch element as the argument, realizing that the results thus obtained may be subject to subsequent modification. In this, the writer follows his predecessors.

NOTATION

The following notation is used in the paper:

t = thickness of arch, in feet.
r = mean radius of arch, in feet.
R_u = radius of extrados, in feet.
R_a = radius of intrados, in feet.
y = ordinate of any point on the arch axis.

ϕ = angle described by any radial line moving from the crown toward the abutment.
ϕ_1 = one-half the central angle, in degrees.
σ_c = cylinder stress, in pounds per square inch.
σ_r = rib-shortening stress, in pounds per square inch.
σ_T = stress corresponding to a drop in temperature.
H_T = normal pull due to a drop of temperature per foot width of arch, at the crown.
H_r = normal rib-shortening at the crown per foot width of arch.
p' = normal radial pressure, in pounds per square foot, on extrados.
T = temperature drop, in degrees Fahrenheit.
n = number of laminæ in the arch dam under consideration.
e = coefficient of thermal expansion.
E = modulus of elasticity of arch, in pounds per square inch.
C = coefficient determined from Figs. 1 and 2 (see Equation (5)).
K = coefficient determined from Figs. 3 and 4 (see Equation (7)).

ANALYSIS OF STRESSES IN CIRCULAR ARCHES BY MEANS OF CHARTS

From formulas and diagrams[4] by William Cain, M. Am. Soc. C. E., it is shown that, other things being equal, the rib-shortening and temperature stresses are determined by the central angle, 2 ϕ_1, and the ratio, $\frac{t}{r}$, of the arch thickness to the mean radius. It is evidently desirable to use such values of these quantities, in so far as may be practicable, as will give minimum values of these stresses, and, hence, maximum efficiency of the arch.

The normal rib-shortening pull at the crown, per foot width of arch, is given by the equation,[5]

$$H_r = -\frac{K'}{12}\, p'\, R_u \left(\frac{t}{r}\right)^2 \qquad \text{(1)}$$

wherein, $K' = \frac{2 \sin \phi_1}{D_s}$.

Likewise, the equation for cylinder stress is,

$$\sigma_c = \frac{p'\, R_u}{144\, t} \qquad \text{(2)}$$

Transposing algebraically to determine a value of $p'\, R_u$ and substituting this value in Equation (1),

$$H_r = -12\, K'\, \sigma_c\, t \left(\frac{t}{r}\right)^2 \qquad \text{(3)}$$

The rib-shortening stress at any section may be expressed by,

$$\sigma_r = -\frac{H_r \cos \phi}{144\, t} \pm \frac{H_r\, y\, r}{24\, t^2} \qquad \text{(4)}$$

Substituting the value of H_r from Equation (3), combining and reducing, Equation (4) becomes,

$$\sigma_r = \sigma_c \frac{K'}{12} \left(\frac{t}{r}\right)^2 \left[-\cos \phi \pm 6\, y\, \frac{r}{t}\right] = C\, \sigma_c \qquad \text{(5)}$$

---

[4] "The Circular Arch Under Normal Loads," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233; see, also, "Design and Construction of Dams," by Edward Wegmann, M. Am. Soc. C. E., Eighth Edition, 1927, chapter on "Multiple-Arch Dams."

[5] "Design and Construction of Dams," by Edward Wegmann, M. Am. Soc. C. E., Eighth Edition, 1927, Equation (9), p. 445.

thus giving the rib-shortening stress in terms of the cylinder stress. For a drop in temperature, the normal pull at the crown per foot width of arch is given by the equation,[6]

$$H_T = -\frac{K'}{12}\, E\, e\, T\, t \left(\frac{t}{r}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

In this equation, T is considered positive, E is in pounds per square foot, and t is in feet. After reducing E to pounds per square inch this expression is identical with Equation (3), except that σ_c is replaced by $E\,e\,T$. The equation for temperature stress at any section may be developed to read,

$$\sigma_T = E\, e\, T \frac{K'}{12}\left(\frac{t}{r}\right)^2 \left[-\cos\phi \pm 6\, y \frac{r}{t}\right] = C\, E\, e\, T \ldots\ldots (7)$$

in which, C is the same as in Equation (5). In other words, after certain simple quantities have been computed, the same coefficient, C, is applicable to determine both the rib-shortening and temperature stresses.

Values of C corresponding to extrados and intrados at both crown and abutment have been computed and plotted on Figs. 1 and 2. It is believed that these curves present the required values in such form as to give a clear picture of the influence of the variables, over a wide range.

It will be observed that when $\frac{t}{r} = 0$, the value of C is zero for all values of $2\,\phi_1$. In other words, as $\frac{t}{r}$ approaches zero, the stress approaches the simple cylinder stress. It is interesting to note also that (within the range of the curves), except for very large angles, the coefficient, C, and, hence, the stress, increases as $\frac{t}{r}$ increases from zero up to some point, and then decreases with further increase in $\frac{t}{r}$. In some cases, and for the smaller angles, the maximum value of the stress corresponds to relatively small values of $\frac{t}{r}$. Also, for some values of $\frac{t}{r}$, the stress may be smaller for smaller angles. Both these facts are perhaps contrary to what would be expected. In other words, the curves show that not for all conditions the rib-shortening and temperature stresses are decreased by decreasing the arch thickness and increasing the central angle.

The sign of the coefficient, C, determines the sign of the rib-shortening stress, σ_r, and also that of the temperature stress, σ_T, for a drop in temperature, which generally is the only direction that need be considered. The total stress due to load is evidently equal to,

$$\sigma_c + \sigma_r = \sigma = (1 + C)\,\sigma_c \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Obviously, there is tension on the section when C is negatively greater than — 1.0, so that the horizontal representing this value on the charts defines the limiting conditions for absence of tension due to load forces.

---

[6] "Design and Construction of Dams," by Edward Wegmann, M. Am. Soc. C. E., Eighth Edition, 1927, Equation (14), p. 448.

From the standpoint of design, it may sometimes be advisable to fix σ and to secure such values of $\frac{t}{r}$ and $2\,\phi_1$ as will accord with this. To satisfy this condition, the value of σ_c must equal $\frac{\sigma}{(1 + C)}$, which must be obtained by trial.

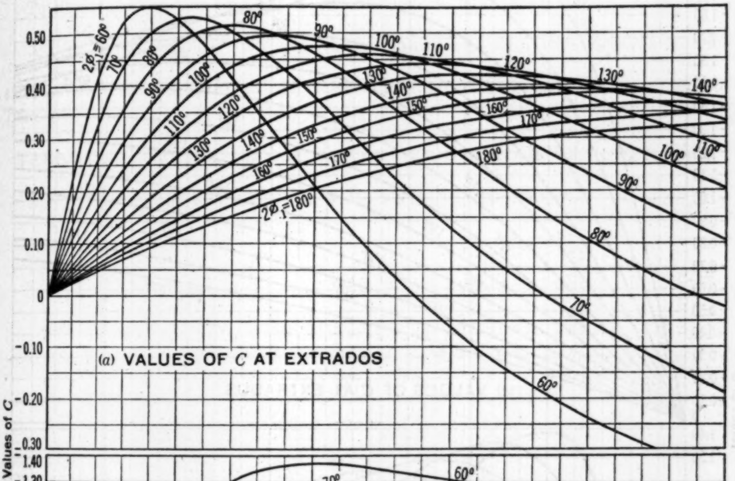


FIG. 1.—VALUES OF COEFFICIENT, *C*, FOR CROWN SECTIONS.

An inspection of the curves will show that the maximum compressive stress will occur at the abutment intrados (Fig. 2(*b*)), and the minimum compression or maximum tension at the abutment extrados. Fig. 2(a) shows that for even moderately large angles and fairly small values of $\frac{t}{r}$, the tensile stresses, due to load only, may become very large. These would be further increased, of course, by the temperature stresses, so that the combined stress may readily become critical.

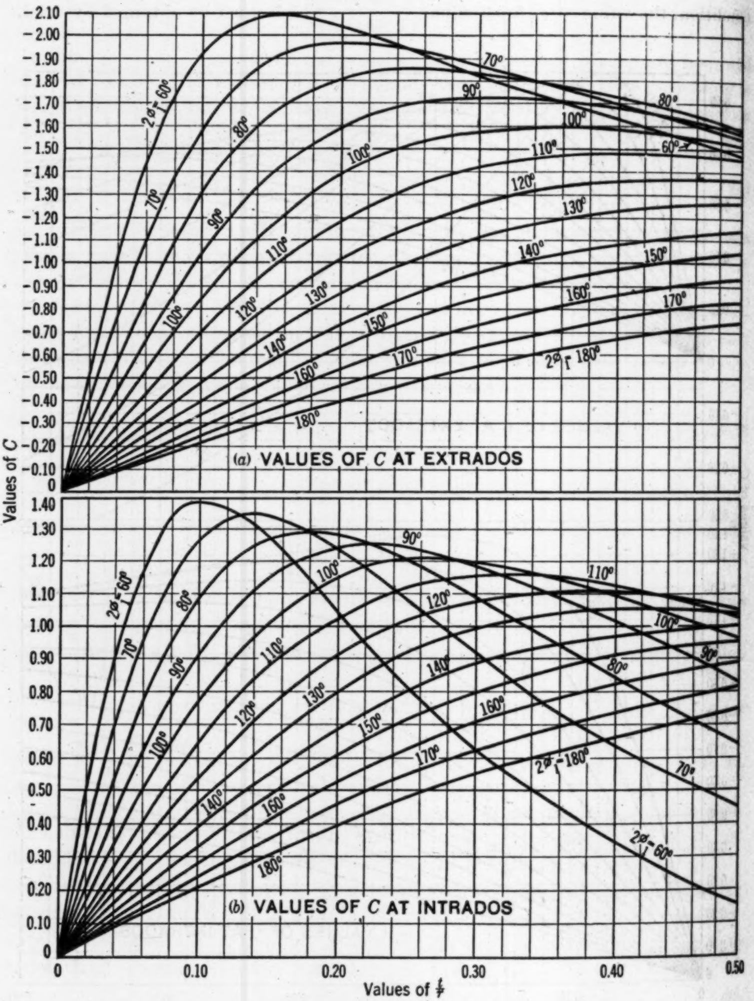


FIG. 2.—VALUES OF COEFFICIENT, C, FOR ABUTMENT SECTIONS.

The direct stress is equal to σ_c. The rib-shortening stress is equal to $C\,\sigma_c$ and the temperature stress is approximately equal to 15 $T\,C$. The coefficient, 15, corresponds to $E = 3\,000\,000$ lb. per sq. in., and $e = 0.000005$, which are probably reasonable average values. In the Stevenson Creek Test Dam an average value of $E = 3\,600\,000$ lb. per sq. in. was found.[7] The change of temperature, T, is expressed in degrees Fahrenheit (negative for a drop).

To illustrate the use of Figs. 1 and 2, assume that $\sigma_c = 350$, $2\,\phi_1 = 110°$, $\frac{t}{r} = 0.10$, and $T = 10°$ (drop). It is required to find the stresses at the extrados and intrados, at crown and abutment. The solution is given in Table 1.

TABLE 1.—COMPUTATIONS TO DETERMINE STRESSES WHEN $\sigma_c = 350$, $2\,\phi_1 = 110°$, $\frac{t}{r} = 0.10$, AND $T = 10°$ DROP.

Section. (1)	Cylinder stress, in pounds per square inch. (2)	Coefficient, C. (3)	Rib-shortening stress, in pounds per square inch = Column (2) × Column (3). (4)	Total load stress, in pounds per square inch = Column (2) + Column (4). (5)	Temperature stress, in pounds per square inch, = 15 × 10 × Column (3). (6)	Total stress, including temperature, in pounds per square inch. (7)
Crown, extrados......	350	0.31	108	458	46	504
Crown, intrados.......	350	−0.39	−136	214	− 58	156
Abutment, extrados..	350	−0.70	−245	105	−105	0
Abutment, intrados...	350	0.65	227	577	97	674

The writer is aware that the curves in Figs. 1 and 2 have been preceded by others, devised with essentially the same purpose, and serving it very well. From his point of view, however, the curves herewith present the facts in a somewhat simpler and more directly usable form, particularly in that the coefficient which they give is applicable both to rib-shortening and temperature stress. This reduces the number of charts and functions with which the designer must deal.

The coefficients, C, can be obtained from the curves without difficulty, with a probable error of not more than 1% or 2% of the higher values. The accuracy is, therefore, sufficient both for the purpose of design and analysis of stresses in arches, of given dimensions. They were computed on the basis of the theory given by Professor Cain.[8] The range of the coefficients, C, in Figs. 1 and 2, is from about -2.1 to $+1.4$ which is very convenient for practical computations.

The coefficients, K, for "thick" arches, were also computed on the basis of Professor Cain's theory.[9] These values were plotted in Figs. 3 and 4. It should be noted, however, that the values of K correspond to $(1 + C)$ in Figs. 1 and 2. The total load stresses, therefore, are obtained directly by the relation: Total load stress is equal to K times the cylinder stress.

---

[7] *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 155.

[8] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

[9] *Loc. cit.*, Vol. 90 (1927), p. 522.

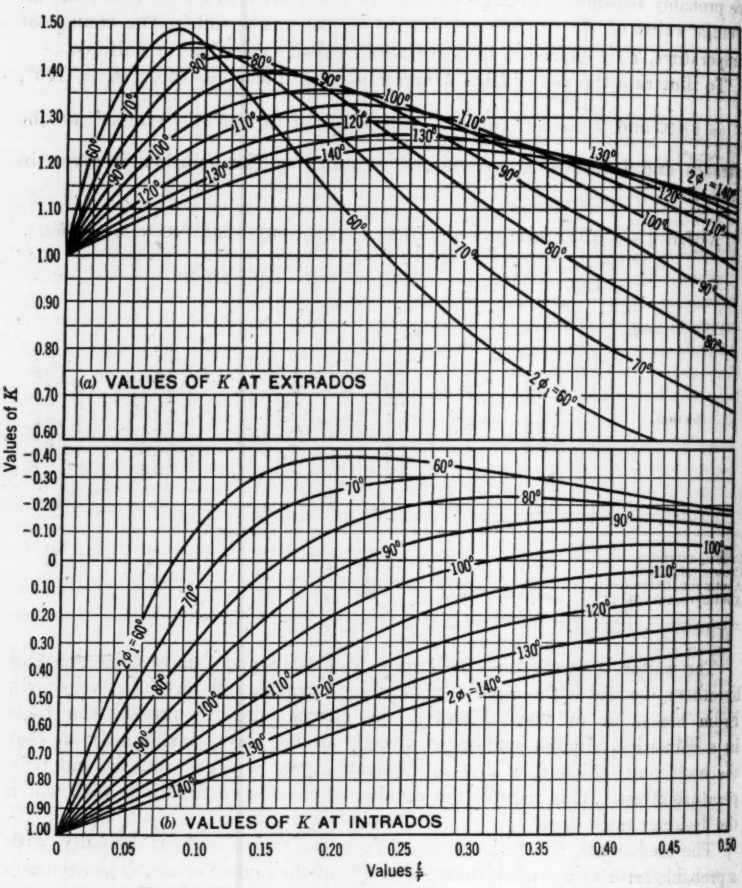


FIG. 3.—VALUES OF COEFFICIENT, K, FOR CROWN SECTIONS.

A comparison of the two sets of diagrams shows that for relatively thin arches there is only a small difference between the values of K and the corresponding values of $(1 + C)$. The difference becomes more marked for thicker arches, that is, for the larger values of $\frac{t}{r}$. For designing purposes it is advisable to use Figs. 3 and 4 to find values of K for thick arches. For computing the stresses due to temperature changes, Figs. 1 and 2 may be used with close approximation.

Laminated Cone Arch Dam with Forked Abutments

The proposed structure to which reference is taken, is shown in Fig. 5 in a typical layout. It is called Dam No. 1 and is a laminated cone arch dam with forked abutments. This structure has been developed in the pursuit of

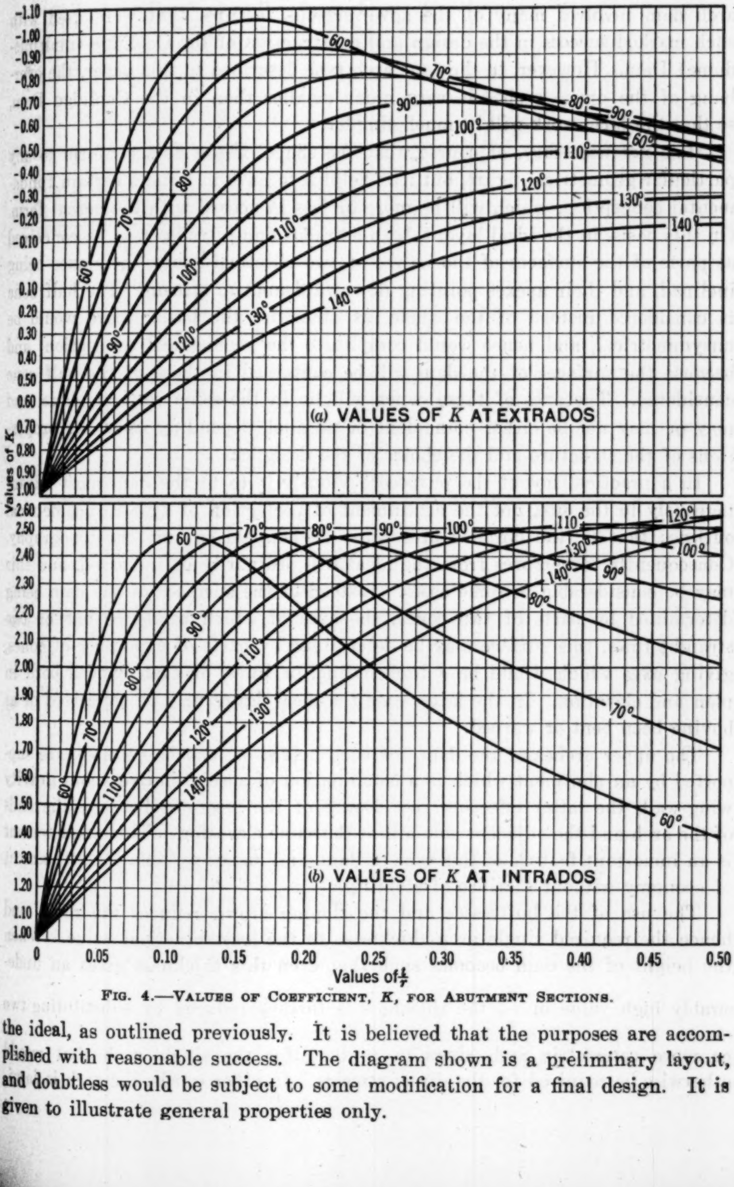


Fig. 4.—Values of Coefficient, *K*, for Abutment Sections.

the ideal, as outlined previously. It is believed that the purposes are accomplished with reasonable success. The diagram shown is a preliminary layout, and doubtless would be subject to some modification for a final design. It is given to illustrate general properties only.

For simplicity of layouts, and because this is sufficient for preliminary design, the arch sections are made circular in horizontal planes, with radii decreasing from crest to foundation. The crown of the dam is given a pronounced slope up stream below the crest, making this section overhang down stream from its foundation. The up-stream slope of the arch crown of a cone arch dam involves many of the advantageous features that were used with such marked success in the design and construction of the Coolidge (multiple-dome) Dam. However, in the cone arch type described in this paper, the overhang of the upper arches is very much smaller than in the Coolidge Dam, so that the form work will be much simpler.

It is not necessary, of course, that the shape of the dam conform to any particular geometrical figure, but it is helpful to have in mind a certain fundamental conception, which will be more or less in accord with the actual form. On this basis, in the ideal layout, both extrados and intrados may be conceived as parts of the surfaces of the halves of two elliptical cones, their axes being inclined, and their apexes pointing downward and up stream. The half cone is considered instead of the whole, as otherwise the basic figure would be unsymmetrical, and hence would complicate the geometrical conception, and because the surfaces of the dam will be contained in those of the half cone considered. The axes of these cones will lie in the same vertical plane, and may or may not be either coincident or parallel, depending upon the proportions of the structure and the choice of the designer.

In a practical layout the purpose obviously is to fit the dam most advantageously to the site, not the attainment of perfection of type in geometrical outline; that is, the circular arcs must be adapted to suit the topography. Consequently, departures from the ideal will ordinarily be indicated, and this may be considerable in some cases. Instead of the surfaces of the dam being determined by parts of two cones, the axes of which would be one or two straight lines, this surface may be determined by parts of a number of cones, giving axes which would be a broken line, or lines, or even curved both in plan and elevation. In the latter case the ideal figure may be conceived of as having been bent or warped.

The upper arches of the dam, for such height as may be suitable, are supported by an abutment which is a combination of special buttress and gravity wing-wall, the latter effecting the closure of the reservoir between the ends of the arch and the adjacent side hill at the same elevations. This arrangement is an important feature of this type of dam, and makes possible the attainment of economy without sacrifice of stability.

The use of the buttresses and the sloping crown reduces the radii, and hence the required single arch thickness, to the least practical value. When the height of the dam becomes such that even this thickness gives an undesirably high value of $\frac{t}{r}$, the thickness is further reduced by substituting two or more concentric arch rings in contact, for the single arch that would otherwise be required for the direct stresses. In other words, the arch is lami-

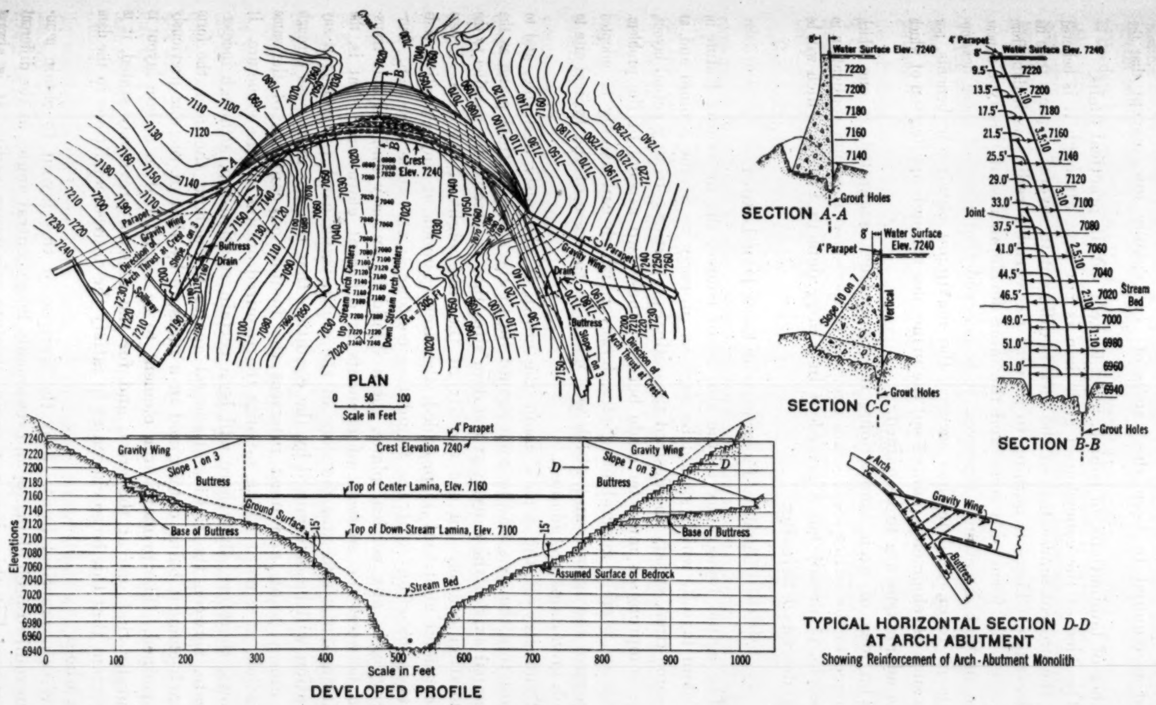

PLAN
0 50 100
Scale in Feet
Crest
Crest Elev. 7240
Up Stream Arch Centers
Down Stream Arch Centers
R = 305 Ft.
Parapet
Gravity Wing
Direction of Arch Thrust at Crest
Slope 1 on 3
Buttress
Drain
Spillway
Creek
A
B
C
SECTION A-A
Water Surface Elev. 7240
Grout Holes
SECTION C-C
4' Parapet
Water Surface Elev. 7240
Slope 10 on 7
Vertical
SECTION B-B
Water Surface Elev. 7240
Joint
Stream Bed
1:10
2:10
2.5:10
3:10
3.5:10
9.5'
13.5'
17.5'
21.5'
25.5'
29.0'
33.0'
37.5'
41.0'
44.5'
46.5'
49.0'
51.0'
51.0'
TYPICAL HORIZONTAL SECTION D-D AT ARCH ABUTMENT
Showing Reinforcement of Arch-Abutment Monolith
Arch
Gravity Wing
Buttress
DEVELOPED PROFILE
4' Parapet
Crest Elevation 7240
Top of Center Lamina, Elev. 7160
Top of Down-Stream Lamina, Elev. 7100
Gravity Wing
Slope 1 on 3
Buttress
Base of Buttress
Ground Surface
Stream Bed
Assumed Surface of Bedrock
15'
D
Elevations
Scale in Feet
0
100
200
300
400
500
600
700
800
900
1000

FIG. 5.—DAM NO. 1, CONE ARCH TYPE.

nated as required to keep the values of $\frac{t}{r}$ acceptably low, except that the number of laminations, of course, must be kept within practical limits.

The idea of the cone-shaped arch dam is not new, although it is believed that the present application of the idea is different in several respects from its predecessors. There is nothing to prevent the variable radius arch dam from assuming the form of a cone, and the later developments of this type appear to be in that direction. However, it is believed that none of these developments of form, to date, has secured the maximum obtainable benefits, nor have such developments pointed out the principles through application of which these advantages are to be gained.

The idea of arch laminations, in the manner suggested, is one which occurred to the writer some years ago, but in the sense of actual application in principle, it already has a precedent in a large dam in France, which will be fully described hereafter.

GENERAL PRINCIPLES OF CONE DAM LAYOUT

The actual topography of the site is, of course, the governing feature in any dam layout, which should be such as properly to fit the ground and, at the same time, satisfy the desired theoretical properties. In any actual layout, some compromises must be made between these two aspects of the problem. There are certain general principles, however, which can usually be adopted to control the layout, and which will do so in the case of the typical site to which arch dams are adapted.

Symmetry of the arches about the crown is obviously desirable, and to secure this the axis of the cone should lie in the same vertical plane as the principal axis of the canyon at the dam site; that is, essentially parallel to the foundation contours in the vicinity of the mid-height of the dam. Unless an artificial abutment is provided, the arch element at any elevation ends at its intersections with the foundation contour of the same elevation. Furthermore, in any vertical plane, there should be an easy and suitable transition between the surfaces of elements at different elevations; that is, the position of the axis and the slope of the crown being fixed, the radius at any elevation will be determined by the condition that the arc must pass through the crown point, and must intersect the foundation contours of the same elevation, at a point which satisfies the required outline of the structure. It follows, therefore, that there will be a certain relationship, although indeterminate, between the radii and central angles throughout the dam, the form of each element being determined in a measure by that of the one previously determined. The primary arch element to be established in a dam layout is naturally that one at the crest, and for the reasons previously stated, it is important that this be so chosen as to satisfy the desired conditions in the dam as a whole.

Within practical limits, it is the purpose of this layout to obtain minimum radii of arch elements and reasonably large central angles at the different elevations, but this does not mean absolute minimum radii. It is always possible to decrease the radii by increasing the central angle, but beyond a certain point this would be wasteful. Furthermore, if the radii in the bottom of the dam become too small, the shape of the corresponding arches is not acceptable. Accordingly, the crest arch element should have a fairly large radius, consistent with a central angle of about 80 to 100° between the controlling abutment points.

FUNCTIONS OF FORKED ABUTMENTS

The primary function of the forked abutments is to decrease the span, and hence the radius that otherwise would be required for the crest and other upper portions of the arch. This should be done without introducing any uncertainties of structural action, or failing to provide in full for the requirements of an arch abutment. Incidentally, the abutments promote symmetry of the arches, which is very desirable, as indicated by the tests on the Stevenson Creek Experimental Arch Dam.

Generally, the two forks of the abutment are placed in such directions relative to the direction of the arch thrust that the thrust is divided into two components supported by the buttress proper and the gravity wing-wall, respectively. The topography at the dam site is usually the guiding factor for the best feasible arrangement of the two forks of the abutment.

For the purposes of design, the buttress is assumed to support its component of the reaction of the abutting horizontal arches. The thickness of the buttress at any elevation is constant and the same as the end of the arch at that elevation. Consequently, the buttress may be considered to be formed by extensions of the horizontal arches.

To insure safety of the buttress against sliding, in addition to the ordinary construction precautions, the layout of the dam should be such that the base of the buttress slopes up the hillside from the end of the arches. From the same point, the top of the buttress may slope downward at the rate of 2:1, 3:1, or any other suitable slope warranted by the conditions.

The height of the buttress, of course, is determined by judgment and economy, but in the writer's opinion there is one consideration which may be given some weight, providing an additional safety factor.

Since the crown of the dam is sloping, the actual load-carrying arch elements will be inclined, and hence only a part of the arch load above the bottom of the buttresses will be transmitted to them. The exact inclination of the true elementary arches cannot be given as a generality, but it will probably be approximately correct to assume them as normal to the slope of the arch crown. If the latter is taken at 4:10, the slope of the normal arches will be 10:4 with the horizontal. The height of the buttress can then be made such that the top inclined arch element (through the crown), if produced, would intersect the base of the buttress and thereby abut directly against the rock foundation. Actually, then, the upper portion of the buttress will receive the reaction only from that part of the dam lying above the uppermost inclined elementary arch. The stability of the buttress, therefore, would appear to be established beyond question, if its down-stream slope is as flat or flatter than, say, 2:1. In fact, the horizontal arch elements abutting against the buttresses will act more or less monolithically with the buttress. The buttress could evidently not be overturned or otherwise overstressed without the entire arch-buttress monolith functioning. Horizontal keyways in the planes of the vertical joints in the arch and between arch and buttresses will tend further to make monolithic action positive. Steel reinforcement extending from the lower parts of the arch upward toward and into the buttresses would give an additional assurance of monolithic action, but such reinforcement, in general, is not necessary.

That the arches of arch dams function more or less monolithically is clearly proved by the fact that recently in two arch dams one abutment was washed away leaving the upper arches without direct support. Nevertheless, the arches, as such, did not fail.[10]

The gravity wing-wall, which, together with the buttress proper, forms the forked abutment of the arch, will also effect the closure of the reservoir between arch and side hill. This wall is supporting its share of the direct water pressure by ordinary gravity action and also the component of the arch thrust acting in its direction. The cross-section of the gravity wing adjacent to the arch is usually relatively large and the stresses from gravity action are very small.

The component of the arch thrust acting on the gravity wing-wall, stresses it in an axial direction; that is, perpendicularly to the stresses from gravity action. The gravity wall abuts laterally against the rock foundations, and the arch thrust component in the gravity wall is thereby transmitted directly to the rock. Assuming a properly stepped foundation, a sliding up hill, or an overturning of the gravity wing in a longitudinal direction, due to the arch thrust component, is naturally a rather remote possibility.

Under normal conditions, a relatively small proportion of the arch thrust may be supported by the gravity wall branch of the forked abutment. It is evident, however, that because of its greater cross-section and consequent smaller deformation, the gravity wing-wall would take an increasingly large proportion of the arch thrust if the primary buttress should be subjected to unforeseen overloads such as might cause it, if acting alone, to approach the point of yielding, due either to excessive concrete or foundation stresses.

The gravity wing-wall serves, therefore, with the same amount of material two distinctly different purposes, wherein lies one of the main advantages of the forked type of buttress.

It is true that a so-called gravity tangent may work on the same principle. However, in general, a gravity tangent will not permit of decreasing the span of the arch as much as a forked abutment, except by running it almost parallel to the contour, which is objectionable. Furthermore, a gravity tangent built along a transverse slope is likely to have its base also on a considerable cross-sectional slope. The sliding factor of the gravity abutment would thus be very high. In addition, the arch has a tendency to deflect down stream and, therefore, it exerts an appreciable torsional reaction on the supporting sections of the gravity wing. This action, combined with the shear due to rib-shortening and temperature, results in an additional force upon the gravity wing for which this is not ordinarily designed. In other words, a gravity tangent to an arch dam is probably the weak link, whereas the forked buttresses as here indicated are believed to be the strongest part of the structure.

---

[10] *Engineering News-Record,* Vol. 97, October 14, 1926, p. 616.

EFFECT OF SLOPING CROWN

The arches of the cone type dam are circular in a horizontal plane, but because of the sloping crown of the dam, and in accordance with the law of least work, the actual working elementary arches will be inclined. The effect of this on the probable reaction carried by the buttresses has been noted. It is evident, also, that the inclined elementary arches possess the advantage of carrying their reaction downward into the foundation, instead of horizontally.

The slope of the cone type of dam permits a much greater variation of the arch radii between crest and base than in the ordinary type of variable-radius arch dam. Thus, for a given span and corresponding radius at the crest, the radii of the lower arch elements can be decreased considerably, usually to one-half or less of the length of the radius at the crest. At the same time the central angles of the elementary arches of a cone dam can be kept reasonably large, say, 100 to 110°, or more, resulting in a better distribution of the stresses over the arch section, for reasons previously given.

Another feature of the sloping crown, and consequent overhang of the central part of the dam down stream from the footing, is that this makes the structure particularly suitable as an overflow dam. By confining the overflow to the central part of the crest, the falling sheet of water will be thrown well down stream from the footings, thus reducing the possibility of critical erosion. Depending upon the character of the foundation material and other conditions, it may be permissible to allow the formation of a natural pool; otherwise, a suitable artificial pool may be created by the construction of a down-stream weir.

EFFECT OF LAMINATIONS

The planes of lamination are concentric, are essentially smooth and true, and the surfaces are painted with asphalt, or other suitable joint material, permitting minute movements and adjustments between the separate arch sections. The typical manner in which the structure is laminated is shown in Fig. 5.

The structural functioning of a laminated arch dam is approximately as follows: The full water pressure in the reservoir acts on the up-stream face of the up-stream arch lamina. Under such pressure this first arch lamina will be deflected. The immediate contact with the second lamina will put this one under stress, inasmuch as the up-stream arch can not deflect without deflecting and stressing the second arch. This latter will take a proportionate amount of the water load according to its thickness and other dimensions governing its stiffness and strength. In case of additional arch laminæ, a further division of the water load between the first, second, and the additional laminæ will take place. Thus, neglecting for the present the effect of cantilever action, each arch lamina will take such a proportion of the total water load that the deflections of corresponding points along radial lines will be the same for all arch laminæ.

Assume, for instance, a horizontal slice, 1 ft. high, which may be composed of, say, three arch laminæ of equal thickness and span, as shown in Fig. 6. The radial water pressure will be divided between the three arches inversely in proportion to their deflections under a unit of load. The actual deflections, of course, are the same for all three arches, if the compression of the concrete in a radial direction is neglected.

If the radii, thickness, and central angles of the individual arch laminæ were exactly the same, it is evident that the water pressure would be divided equally between the three arches in order to produce the same deflections.

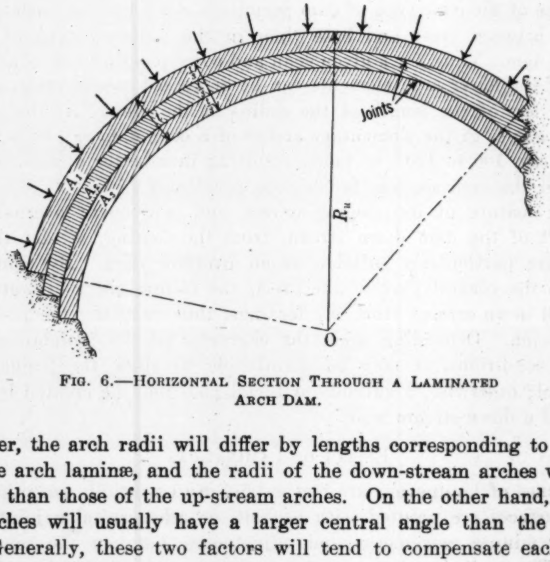


FIG. 6.—HORIZONTAL SECTION THROUGH A LAMINATED ARCH DAM.

However, the arch radii will differ by lengths corresponding to the thickness of the arch laminæ, and the radii of the down-stream arches will always be smaller than those of the up-stream arches. On the other hand, the down-stream arches will usually have a larger central angle than the up-stream arches. Generally, these two factors will tend to compensate each other as regards their influence on the deflection of the various arch laminæ and their ability to carry their proportion of the load.

It will be a relatively simple matter to determine in each case the proportion between the thicknesses of the individual arch laminæ, in order to secure approximately the same stresses in all of them.

The joints between the arch laminæ incidentally will subdivide the vertical cantilevers of the dam into cantilever laminæ. The deflection of a cantilever under a given load is inversely proportional to its moment of inertia. In the case of a cantilever, 1 ft. wide, cut by assumed vertical planes from a dam, the deflection due to a given load is inversely proportional to the third power of the thickness. Then, for a cantilever with n laminæ of equal thickness, and assuming further the total load to be equally divided between the n laminæ, the deflection of each lamina is increased by $\frac{n^3}{n} = n^2$. In other words, the deflection of the laminated cantilever is increased by the square of the number of laminations. Consequently, the cantilever elements of laminated arch dams will be more flexible and the bending moments at the base of the cantilevers which in a monolithic dam would be likely to produce cracks between the dam and the foundation at the up-stream face, are greatly reduced. Furthermore, each cantilever element will be bent individually, thus giving a better stress distribution upon the foundation than in a monolithic structure.

The laminating slightly increases the deflection of the arches. The deflection of the vertical elements is increased in proportion to the square of the number of laminations. A greater proportion of the load, therefore, is thrown upon the arches, especially near the base of a dam; this is desirable because it increases the efficiency of arch action.

The effectiveness of laminating the arches of thick arch dams is evident from the following considerations. Suppose it were practicable to subdivide an arch dam having a radius of 100 ft. into a number of very thin concentric arch laminæ, say, 1 ft. in thickness, each. The dam would then be composed of extremely thin arch rings for which the cylinder formula would be applicable with a relatively small percentage of error. The vertical cantilever elements would be so thin and flexible that they would probably carry only a very small percentage of the load except in the immediate vicinity of the foundation. The cylinder formula, therefore, would be approximately valid for such a structure and the water pressure would be transmitted to the arch abutments by a practically uniform axial compression in the thin arch laminæ. From the theoretical point of view this would result in maximum efficiency, and for a given allowable unit stress would require a minimum of concrete. In practice, this ideal, of course, can not be attained, but it can be approached by providing a reasonable number of laminations.

It is evident that the principle of laminating arch dams may be applied also to arches which increase in thickness toward the abutments as well as to arches of uniform thickness between abutments. In fact, arches which are thicker at the abutments will be most economical, because the critical stress is generally at that point.

There are several ways to provide joints for laminations without unduly increasing the unit prices of the concrete of the arch. For instance, the arch on the down-stream side of the joint may be built one lift, say, 5 ft., higher than the adjacent up-stream arch; the up-stream forms of the higher arch may then be removed, and the face painted with asphalt or some other suitable material to prevent bond across the joint. Then, the up-stream lamina is poured directly against the face of concrete so treated.

Instead of a coating of asphalt the up-stream face of the down-stream lamina may be provided with a water-proofing membrane, thus giving reasonable assurance that no water will seep through to the down-stream face of the dam. By placing split drain pipes against the membrane at suitable intervals of, say, 20 to 40 ft., any water seeping through the up-stream arch lamina may be drained off readily.

A simple method of producing a cold joint between the arch laminæ, would be to place and securely hold in position, a continuous sheet of impregnated burlap or elastite and then pour concrete on both sides of the sheet. This will provide a joint, at a minimum interference with concreting.

It is important that the lamination of an arch dam be arranged so that the highest lamina is at the up-stream side of the dam. The other laminæ are placed in steps down stream from the first lamina. The laminæ are not connected with each other, except possibly along the foundation. Any water that might seep from the reservoir through the first arch into the joints between the laminæ, can easily be drained away, if desired. One may also choose to permit a film of water to accumulate in the joints between the different arch laminæ so that each lamina would be subjected to a definite hydrostatic pressure. Any surplus water in the joints would be likely to escape from them over the tops of the different laminæ, or could be drained in a lateral direction at any desired elevation toward the arch abutments. In any case, it is quite obvious that none of the individual arch laminæ could ever be overstressed by excessive hydrostatic pressure in a joint.

Fig. 7 shows the plan, elevation, and cross-sections of a laminated cone arch dam proposed to be built for an irrigation project in California. In order to insure positive and unfailing action of the drainage system in the joint between the arch laminæ a number of overflow openings are provided along the top of the lower arch lamina. Water rising in the joint to the top of the laminæ would immediately drain off and thus prevent the lower arch lamina from being subjected to excessive hydrostatic pressure. In addition to these overflow openings, drains are provided in the joint leading to an inspection and drainage tunnel extending along the foundation of the arch. From this inspection tunnel the vertical drains in the joint may be opened up, if necessary. Thus, there is no possibility of the down-stream arch lamina ever becoming overloaded through clogging of the drains.

The factor of safety of a laminated arch dam can be ascertained by model tests in a relatively simple manner by loading each arch lamina of the model separately to the point of breaking. The difficulty and expense of securing sufficient static pressure to produce failure of each arch lamina separately, are evidently much smaller than if a model of the complete dam were tested to destruction.

An investigation was made to determine whether the friction in the asphalted joint might prevent the sliding of the arch laminæ upon each other, thus defeating to some extent the purpose of laminating. Assume an elementary horizontal arch with two arch laminæ at a depth of 200 ft. below the crest of a dam. The water pressure at this depth is $62.5 \times 200 = 12\,500$ lb. per sq. ft. The up-stream lamina is assumed to carry one-half this pressure, the other half (6 250 lb. per sq. ft.) being transmitted across the joint to the down-stream lamina. Assuming a coefficient of friction in the asphalted

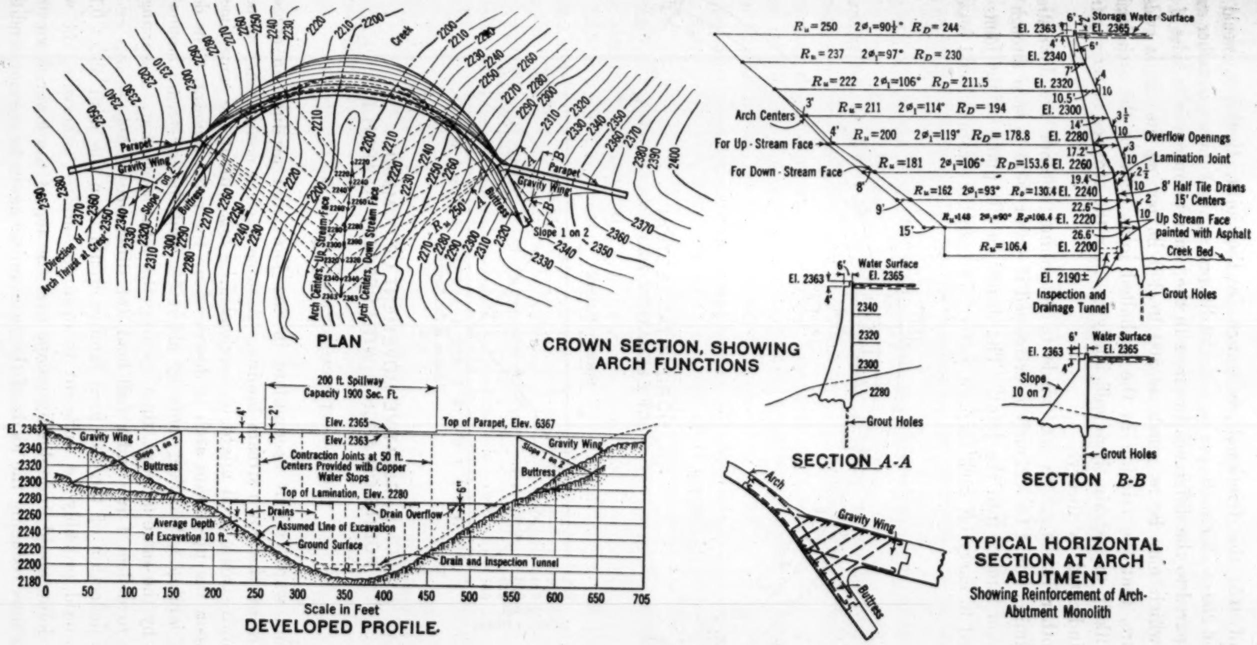
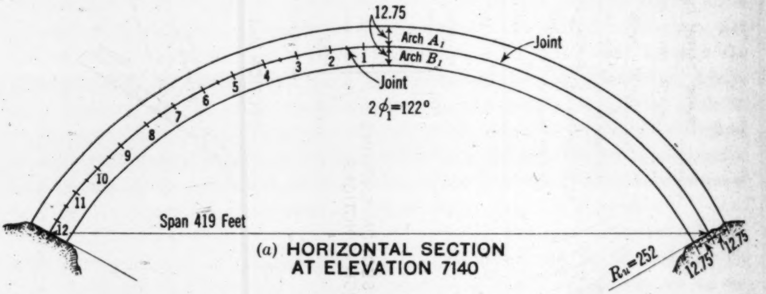

PLAN
Parapet
Gravity Wing
Slope 1 on 2
Buttress
Direction of Arch Thrust at Crest
Creek
Arch Centers, Up Stream Face
Arch Centers, Down Stream Face
A
B
R_u = 250
CROWN SECTION, SHOWING ARCH FUNCTIONS
R_u = 250 $2\phi_1$ = 90½° R_D = 244
R_u = 237 $2\phi_1$ = 97° R_D = 230
R_u = 222 $2\phi_1$ = 106° R_D = 211.5
R_u = 211 $2\phi_1$ = 114° R_D = 194
R_u = 200 $2\phi_1$ = 119° R_D = 178.8
R_u = 181 $2\phi_1$ = 106° R_D = 153.6
R_u = 162 $2\phi_1$ = 93° R_D = 130.4
R_u = 148 $2\phi_1$ = 90° R_D = 106.4
R_u = 106.4
Arch Centers
For Up-Stream Face
For Down-Stream Face
3'
4'
8'
9'
15'
El. 2363
El. 2340
El. 2320
El. 2300
El. 2280
El. 2260
El. 2240
El. 2220
El. 2200
El. 2190±
6'
7'
10.5'
14'
17.2'
19.4'
22.6'
26.6'
Storage Water Surface
El. 2365
Overflow Openings
Lamination Joint
8' Half Tile Drains 15' Centers
Up Stream Face painted with Asphalt
Creek Bed
Inspection and Drainage Tunnel
Grout Holes
SECTION A-A
Water Surface
El. 2365
El. 2363
2340
2320
2300
2280
Grout Holes
SECTION B-B
Water Surface
El. 2365
El. 2363
Slope 10 on 7
Grout Holes
TYPICAL HORIZONTAL SECTION AT ARCH ABUTMENT
Showing Reinforcement of Arch-Abutment Monolith
Arch
Gravity Wing
Buttress
DEVELOPED PROFILE
200 ft. Spillway Capacity 1900 Sec. Ft.
Top of Parapet, Elev. 6367
Elev. 2365
Elev. 2363
Contraction Joints at 50 ft. Centers Provided with Copper Water Stops
Top of Lamination, Elev. 2280
Drains
Drain Overflow
Average Depth of Excavation 10 ft.
Assumed Line of Excavation
Ground Surface
Drain and Inspection Tunnel
Gravity Wing
Buttress
Slope 1 on 2
El. 2363
2340
2320
2300
2280
2260
2240
2220
2200
2180
0
50
100
150
200
250
300
350
400
450
500
550
600
650
705
Scale in Feet

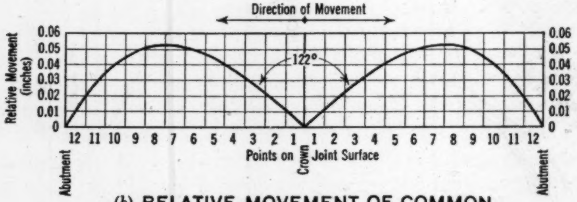
FIG. 7.—PLAN AND SECTIONS OF DAM NO. 2.

joint of 0.5, the frictional resistance in the joint, therefore, would be 0.5 × 6 250 = 3 125 lb. per sq. ft. (22 lb. per sq. in.) This is rather small as compared to the difference in stress in the concrete on opposite sides of the joint, which may be as much as 200 to 300 lb. per sq. in. It is evident, therefore, that the friction in the asphalted joint between the arch laminæ is not likely to restrain them sufficiently to alter the stresses materially from those indicated by theory.

Another investigation was made to determine whether the down-stream arch lamina could be seriously overstressed if water were to seep through the up-stream lamina into the joint. The drains in the plane of the joints are provided to carry off such seepage water. In case the drains should cease to function, water might accumulate in the joint and produce an excess of pressure on the down-stream lamina. Assume such a contingency to occur. The down-stream arch might be overloaded by 5%, but at the same time the load upon the up-stream arch is decreased by the same amount. The down-stream lamina will deflect more by about 5% and the up-stream lamina will recede by the same amount, thus opening the joint somewhat. Assuming the arches to deflect 1 in. under full load, the difference of 5% + 5% = 10% of the deflection of the two arch laminæ would open the joint about 0.1 in. This would probably be sufficient to open the drains; otherwise, the water in the joint would undoubtedly escape over the top of the lower down-stream lamina thus reducing the loads of the two laminæ again to normal conditions.



(*a*) HORIZONTAL SECTION AT ELEVATION 7140



(*b*) RELATIVE MOVEMENT OF COMMON POINTS ON ARCH A_1 WITH RESPECT TO ARCH B_1

FIG. 8.—TYPICAL ARCH OF DAM NO. 1.

In case of differences of the temperature of the up-stream and down-stream laminæ the dam would not be any worse off than a monolithic arch. In fact, grouting of the vertical joints of the laminæ with different pressures would afford a convenient means for correcting subsequent influences of temperature variations.

DETERMINATION OF MAXIMUM ARCH THICKNESS

The maximum allowable arch thickness, and hence the number of laminations required at any elevation, depends, of course, on the design assumptions and working stresses. Assuming the preliminary design to be based on a uniform stress of 350 lb. per sq. in. by the cylinder formula, and assuming the actual critical stress to be the tension at the abutment, it is indicated that the maximum uniform thickness is reached when $\frac{t}{r}$ is about 0.15, assuming $2\phi_1$ at about 90 to 100 degrees. Recourse to laminating is not necessary at this point, however, as the critical stress can be reduced by thickening the arch at the abutment. The point at which the first or subsequent laminations are required, is, therefore, a matter of judgment dependent upon the conditions, to a certain extent.

Fig. 8 (*a*) is a typical horizontal section through the dam shown in Fig. 5 at Elevation 7 140. The arch at this elevation is thicker at the abutments than at the crown, but in order to simplify the computations for the present purpose, it will be assumed that the arch laminæ are of uniform thickness from abutment to abutment.

Table 2 gives the stresses at the crown and abutments in the two arch laminæ and, by way of comparison, also in the unlaminated arch.

TABLE 2.—COMPARISON OF ARCH STRESSES.

Arch. (see Fig. 8.)	Arch thickness, t, in feet.	Radius at center line, r, in feet.	$\frac{t}{r}$.	Central angle, $2\phi_1$, in degrees.	ARCH STRESSES, IN POUNDS PER SQUARE INCH.								
					Cylinder.	Rib-Shortening.		Cylinder Plus Rib-shortening.		10° Drop of Temperature.		Total for Load and Temperature.	
						Crown.	Abutment.	Crown.	Abutment.	Crown.	Abutment.	Crown.	Abutment.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
A_1, Extrados.......	12.75	245.6	0.052	122	429	64	−129	493	300	25	−50	518	250
A_1, Intrados	12.75	245.6	0.052	122	429	− 70	125	359	554	−27	48	332	602
A_2, Extrados.......	12.75	232.9	0.055	122	429	67	−135	496	294	26	−52	522	242
A_2, Intrados	12.75	232.9	0.055	122	429	− 75	131	354	560	−29	50	325	610
Monolith, extrados.	25.50	239.3	0.106	122	429	114	−249	548	180	44	−96	587	84
Monolith, intrados.	25.50	239.3	0.106	122	429	−140	232	289	661	−54	89	235	750

From the values given in Column (14) of Table 2, it is seen that in the arch under consideration the maximum stress at the abutment intrados is

610 lb. per sq. in. in the laminated arch, while in the monolithic arch the maximum stress at the same point would be 750 lb. per sq. in. The process of laminating this arch reduces the maximum stress at the abutments by 140 lb. per sq. in., which is between 20 and 25% in favor of the laminated arch. For laminated arches of non-uniform thickness the reduction in stress, or the economy of material in case of the same size of maximum stress that may be obtained by laminating, is generally about in the same proportion as for arches of uniform thickness, such as those considered in the preceding example.

Fig. 8(*b*) gives the movement of Arch A_1, relative to the arch, A_2, in the plane of the joint between the two arches, due to a change of water pressure from no load to full load. For this purpose the stresses and corresponding strains at the intrados of Arch *A*, and at the corresponding points across the joint, at the extrados of Arch A_2 were calculated and the elastic deformations at the arch surfaces added from abutment to abutment. A modulus of elasticity of 3 000 000 lb. per sq. in. was used in the computations.

Each arch was divided into twenty-four sections of equal length, and the stresses were computed on radial sections through the center of these lengths, as indicated by the numbered points in the diagram. The stresses thus found were assumed to be the average for each increment, ΔS, of length, and the deformation computed accordingly, on the intrados of Arch *A*, and the extrados of Arch *B*. Beginning at the abutment, these deformations for each arch were progressively added. The difference in these sums, for any point, gives the movement of this point on the intrados of Arch *A* with respect to the same point on the extrados of Arch *B*, which is shown graphically in the diagram.

Thus, for a laminated arch of the shape and dimensions shown in Fig. 8(*a*) subjected to stresses, as given in Table 2, the maximum relative movement of the two arches in the plane of the joint would be about 0.05 in. This is very little and may take place in an asphalted joint without difficulty. The maximum sliding movement in the joint of 0.05 in. will occur only in case of a change from no load to full load conditions, or *vice versa*, and accompanied by a change of temperature corresponding to that assumed in Table 2. For most dams, such conditions obtain not oftener than once or twice each year.

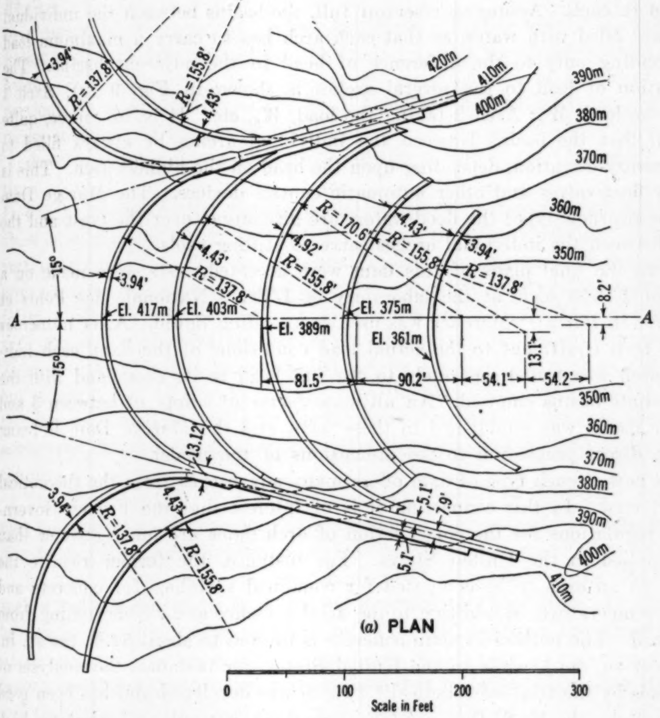
FRENCH DAM UTILIZING PRINCIPLE OF LAMINATION

In connection with the writer's proposed lamination of thick arch dams, an account of a high dam under construction in France, in which the same principle has been utilized, should be of interest. Since this structure represents such a bold departure from precedent it is believed that a fairly complete description will be in order.

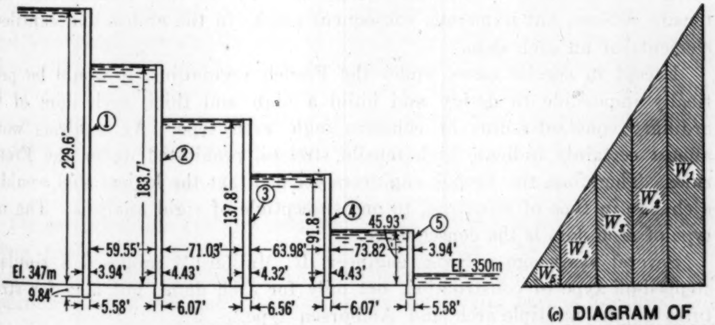
The dam is at Marège, on the Dordogne River, in France.[11] The lamination is effected in a rather different way from that proposed by the writer, but the purpose in view, and except for slight differences, the general structural effects are essentially the same. The dam, as shown in Fig. 9, will be

---

[11] *Revue Générale de l'Electricité*, October 29, 1927.


3.94'
R=137.8'
R=155.8'
4.43'
420m
410m
400m
390m
380m
370m
360m
350m
R=170.6'
4.43'
4.92'
R=155.8'
R=155.8'
3.94'
R=137.8'
4.43'
R=137.8'
3.94'
15°
15°
8.2'
A
A
El. 417m
El. 403m
El. 375m
El. 389m
El. 361m
13.1'
81.5'
90.2'
54.1'
54.2'
350m
360m
370m
380m
13.12'
5.1'
7.9'
390m
400m
410m
3.94'
4.43'
R=137.8'
R=155.8'
5.1'
0
100
200
300
Scale in Feet

(a) PLAN


①
②
③
④
⑤
229.6'
183.7'
137.8'
91.8'
45.93'
59.55'
71.03'
63.98'
73.82'
3.94'
4.43'
4.32'
4.43'
3.94'
El. 347m
El. 350m
9.84'
5.58'
6.07'
6.56'
6.07'
5.58'
W_1
W_2
W_3
W_4
W_5

(b) SECTION A-A

(c) DIAGRAM OF LOAD DISTRIBUTION

FIG. 9.—DORDOGNE RIVER DAM AT MARÈGE, FRANCE, PLAN AND SECTIONS.

composed of five thin arches of reinforced concrete, the thickest being only 4.9 ft., and separated by distances varying from about 59.5 to 73.8 ft. The tallest arch will be about 230 ft. high, the others being lower in steps of about 46 ft. each. Assuming reservoir full, the basins between the individual arches are filled with water, so that each arch has to carry a maximum load corresponding only to the difference of head in the adjacent basins. The distribution of load to the several arches is shown in Fig. 9 (*c*), Arch 1 taking the load, W_1; Arch 2 taking the load, W_2, etc. It is, of course, quite essential that the basins between the individual arches be always filled to the necessary elevation, depending upon the head on the highest arch. This is done by float-valves and other automatic control devices. The Marège Dam is of the overflow type; the flood-waters are discharged over the crest and the basins between the individual arches serve as stilling-pools.

Before the final plans of this dam were accepted, tests were made on a model of 1 : 100 scale at the laboratory of L'Ecole Nationale des Ponts et Chaussées in Paris.[12] Mercury was used as a loading liquid. After numerous general tests equivalent to the actual load conditions of the dam, each individual arch was tested separately to the full head to its crest, and with the intermediate basins emptied. An ultimate factor of safety of between 3 and 4 of the model was established in these tests, and the Marège Dam is being built in direct proportion to the dimensions of the model.

This new French type of dam offers many advantages from the theoretical point of view. In this connection it is of interest that the French Governmental regulations for the construction of arch dams are much stricter than those applied in the United States. For instance, the former require the stresses in an arch to be computed for combined shrinkage of concrete and drop of temperature, in addition to the axial pressure and rib-shortening from water load. The tension in plain concrete is limited to about 52 lb. per sq. in. (4 kg. per sq. cm.), while in the United States, for instance, the analysis of arch dams by the trial-load method[13] (which was developed and has been used extensively by the U. S. Bureau of Reclamation), not only contemplates high tensile stresses, but numerous consequent cracks in the arches and cantilever elements of an arch dam.

Except in special cases, under the French regulations it would be practically impossible to design and build a high and thick arch dam of the ordinary constant-radius or constant-angle arch type. An analysis would almost certainly indicate high tensile stresses, prohibited under the French rules. Therefore, the French engineers decided that the logical step would be a change in type of structure, to one susceptible of rigid analysis. The new type of arch dam is the consequence.

Several years ago a Swiss engineer, R. Maillard,[14] proposed a similarly stepped-off type of construction, not only for arch dams, but also for structures of the multiple-arch and Ambursen type.

---

[12] A description of these model tests and a photograph of the model are given in *La Revue Industrielle*, July, 1927, p. 340; see, also, *Engineering News-Record*, August 30, 1928, p. 311, and *Western Construction News*, July 25, 1928, p. 465.

[13] "Analysis of Arch Dams by the Trial-Load Method," by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1191.

[14] *Schweizerische Bauzeitung*, April 14, 1928, p. 183.

The Swiss and French designs probably represent the ultimate attainment in the application of pure logic to the problem of high arch dams, resulting in a design of maximum structural simplicity and determinateness. The striking feature of their plan is that not only is the total load distributed to a series of independent arches, but it is so subdivided that, except the lowest arch, each one is uniformly loaded vertically for the greater part of its height. This, obviously, tends to a great simplification of structural action, particularly when buttresses are used, keeping the length of many arch elements constant, as they have done.

Another outstanding characteristic of this design is the very small foundations which it requires, except in case of a great depth of loose overburden. It will be observed that the aggregate width of foundations of the Marège Dam is only about 30 ft., and this for a dam the effective height of which is very nearly 230 ft. This is, of course, a direct consequence of the very thin arch sections, and the assurance that the vertical foundation reactions will be fairly uniformly distributed. Judging from the topography, in connection with the sectional elevation, the depth of excavation is also very moderate. This, of course, might be accounted for by excellent foundation conditions, but it might also be due to the fact that the maximum argument for leakage through the foundation is only the difference in head on the two sides. This same consideration is an important justification for the relatively extreme thinness of the arches.

The adaptability of this particular type of layout to dam sites in general, may be open to some question, and the plan would probably require modification to suit varying conditions.

In spite of the great apparent theoretical advantages of the design, it is such a radical departure from present practice that engineers elsewhere may be slow to adopt it. Also, contrary to its actual functions, the structure as a whole presents a rather complex appearance, which may be regarded with disfavor; the extremely thin concrete sections may be looked upon with suspicion, and the form work required may be an adverse feature.

The writer's proposed method of lamination, while falling short of the ideal theoretical accomplishments of the French design, perhaps sufficiently achieves the desired results, more harmoniously with American practice and conditions. Particularly from the standpoint of appearance, the finished structure will seem to be a massive monolith, similar in outline to structures representative of advanced American practice.

REDUCTION OF CANTILEVER INFLUENCE IN DESIGN

The several design features proposed by the writer will make it possible to obtain relatively thin arches throughout the dam. As pointed out, the cantilever elements will become comparatively ineffectual. Consequently, the elementary arches will not only take by far the greater part of the load, but this will be distributed fairly well in direct proportion to the water pressures; that is, horizontal elementary arches would carry approximately a uniform radial load. The reason for dividing the total load between arches and the cantilevers, in the case of thick arches, is not merely because this produces refinements of design from the standpoint of economy, but because the cantilever action may result in more unfavorable conditions than if they were absent. In other words, if the cantilever action were neglected, not only would the resulting analysis be entirely erroneous, but it might be actually unsafe. Evidently, the thicker the arch the greater the necessity for considering cantilever action; conversely, the thinner the arch, the greater the assurance that this may be omitted without jeopardizing the safety of the analysis, which will at least approach the actual conditions. While the writer does not favor discarding any proper refinements of analysis, it is believed that in the proposed design, the elementary arches might be assumed to carry the entire water pressure, and yet give essentially the same dimensions of structure as if the cantilevers were considered. At least, the preliminary design would require the minimum of subsequent modification. The design, of course, may be checked and altered if necessary, by the trial-load method,[15] investigating a suitable number and combination of elementary cantilever and arch elements.

To avoid an undesirable reversal of stress at the base of the dam, and to make the action of the vertical elements more determinate, a temporary hinge may be provided during construction, and later grouted, as recommended[16] by Victor H. Cochrane, M. Am. Soc. C. E. This, of course, would tend to diminish the cantilever action even more.

THE RAILROAD CANYON CONE ARCH DAM

As an example of the actual application of the design principles proposed by the writer (except that of lamination), there follows a description of the Railroad Canyon (cone arch) Dam, built in 1927-28, on the San Jacinto River, near Elsinore, Calif. (See Figs. 10 and 11.) The height and layout of the dam were such that laminations were not required. The properties of this design, except laminations, are generally the same as those that have been described and discussed in detail in connection with the typical design, Fig. 5.

The dam is of the overflow type, the spillway over the central portion of the crest being designed for 30 000 sec-ft. The maximum recorded flood at the site is 15 000 sec-ft. The crest overflow will be spilled into a stilling-pool formed by a small arch, 20 ft. high, built about 50 ft. below the dam. (See Fig. 10.) Except immediately adjacent to the crest and the base, the crown slope is 10:4, which gave a layout suitable to the topography.

Those portions of the arches adjoining the abutments, and also both forks of the abutments were strongly tied together with 1½-in. round reinforcing bars. The buttress was designed to take the computed arch thrust, but, as previously stated, it seems evident that failure could occur only by the simultaneous crushing or sliding of both the buttresses and the wing-wall. This affords a very large safety factor, over and above that provided in the design.

The highest axial stresses in the horizontal arches due to the water pressure are about 360 lb. per sq. in. The arches are of uniform thickness at the upper elevations, but are thickened toward the abutments, in the lower portion

---

[15] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1191.

[16] *Loc. cit.*, Vol. LXXXVII (1924), p. 378.

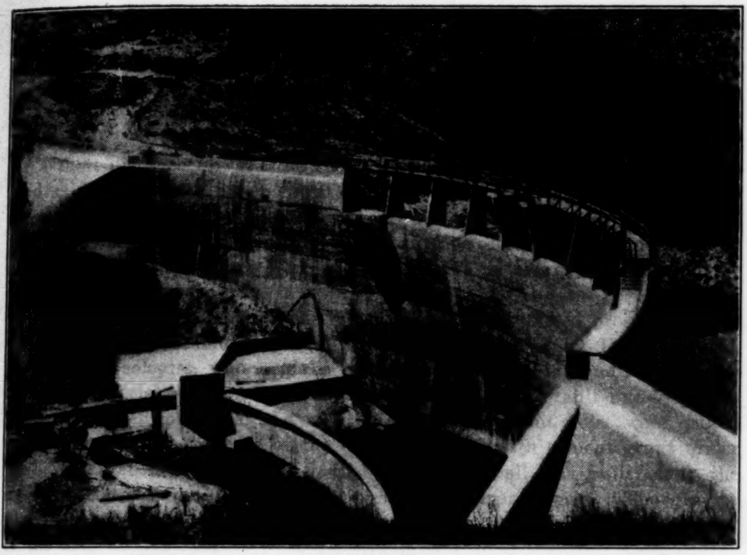


FIG. 10.—DOWN-STREAM FACE, RAILROAD CANYON DAM.

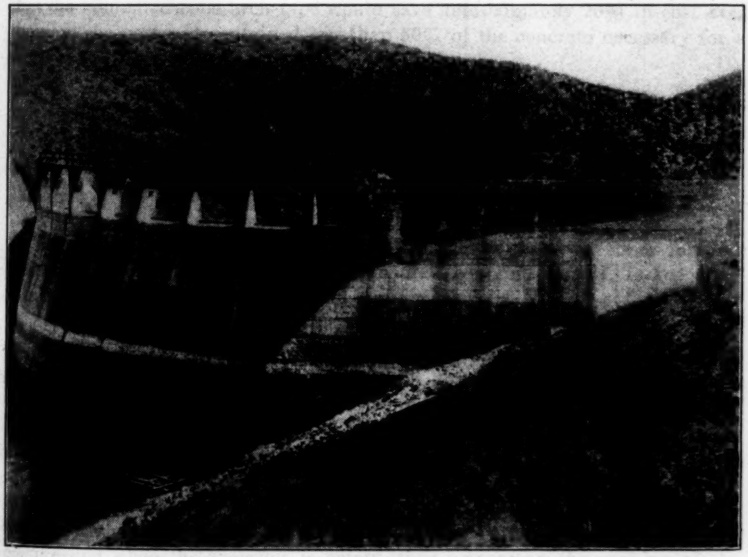
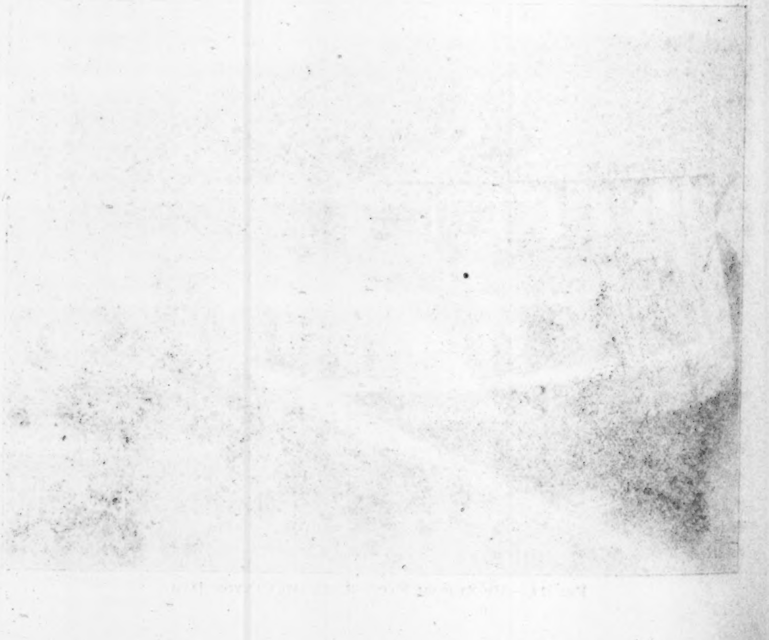


FIG. 11.—UP-STREAM FACE, RAILROAD CANYON DAM.

[illegible]



[illegible]

of the dam. In the design, the maximum allowable compression was taken at 600 lb. per sq. in., and the maximum tension at 100 lb. per sq. in., including the stresses due to rib-shortening, which are the limits set by the State Engineer of California.

The bed-rock at the dam site is slate. In the bottom of the canyon and for a short distance upward the rock is extremely hard, in places almost as hard as flint. At the upper elevations it is softer and fractured into small pieces. Where the shattered rock was encountered the excavation was varied to a maximum of 35 ft. below the original surface of the ground, and the concrete abutments of the arch were widened to give an average pressure on the rock of about 10 tons per sq. ft.

A fault line in, and approximately parallel to, the river bed, crosses the dam site, but this is considered "dead" in an opinion given by Roy G. Mead, Consulting Geologist, who made a report on the geologic conditions at the dam site. Special care was taken in the excavation at the fault line by grouting and deep cut-off work. In addition, numerous grout holes were drilled into the rock below the base of the dam, and filled with cement grout under pressures of about 100 lb. per sq. in. A cut-off wall was built along the up-stream face of the dam for further safety against seepage. The gravity wing-walls were designed for the hydrostatic head assumed to act over 50% of the area of the base.

Before deciding upon the dam for this location, comparative designs and estimates of cost were made for five types of dams. These studies indicated that the laminated cone arch type would save approximately 20% in cost over the next cheapest. It contained less than 50% of the concrete necessary for a straight gravity dam.

ACKNOWLEDGMENTS

The writer is indebted to S. M. Cotten, M. Am. Soc. C. E., for valuable aid in the preparation of this paper.

CONCLUSIONS

The type of arch dam described in this paper involves three features which promote both simplicity of design and economy, namely, forked abutments, sloping crown of the arch, and laminations:

(1) The so-called "forked abutments" take the thrust of the upper portions of the arch. Each abutment consists of a buttress and a gravity wing-wall poured integrally, thereby forming an arch abutment of large cross-section and great resistance to shearing, overturning, and sliding. The forked abutments reduce the span of the arch and permit the use of relatively short arch radii and a correspondingly small thickness of the arch.

(2) A pronounced sloping of the crown of the arch, secures short radii and large central angles for the arch elements below the crest. The small overhang of the arch in the center will keep the upper arch elements under light compression when the reservoir is empty, thus preventing the opening of the construction joints.

(3) The sub-division of the total arch thickness into two or more arch laminæ, keeps the arches relatively "thin" and reduces greatly the cantilever action so that the arch elements are more nearly uniformly loaded and may be designed without recourse to complicated methods of arch analysis.

These three features may be used to advantage either singly, in combinations of two, or all three of them, in the design and construction of arch dams at most sites. The resulting structures will involve arches of high efficiency, subject to simple, but nevertheless comparatively rigid analysis. The forked abutments are structural elements with a high degree of design and reserve strength. They can be arranged at many sites in such a way that they improve greatly the conditions as to span, symmetry, and economy of the main portion of the structure, namely, the arch.

DISCUSSION

Lars R. Jorgensen,[17] M. Am. Soc. C. E. (by letter).—At and toward the bottom of a high arch dam in a narrow canyon the arch action is inefficient, and the dam has much greater stability as a plug, if the excavation is shaped in such a way that shear parallel to the side-hill can be resisted. The concrete has a very large shearing resistance and it is just as well to take advantage of this.

For fairly thin arch dams the writer would prefer to keep the arch solid in order to preserve full shearing strength in the concrete in all directions. Some difficulties may also arise in getting the laminations to slide on one another, and, moreover, this type of construction increases the form work.

A single trial calculation of the unit shear in the lower portion of a thick arch dam in a narrow canyon will, in general, disclose average shearing stresses along the contact area with rock of less than 100 lb. per sq. in., the assumption being that the plug carries full water pressure. The load is actually divided between arch action and punching shear and since the former is inefficient in this zone, most of the load is carried by punching shear, and it is evident that as long as the unit punching shear is low, the thick arch portion cannot fail. While the shear is normally not uniformly distributed over the contact area, it is believed that some time before failure it will be so distributed.

The ultimate shearing strength of concrete in the lower part of a high arch dam should not be less than 2 000 lb. per sq. in. for good concrete, especially since some arch compression acts more or less perpendicular to the plane of shear. A high dam in a narrow canyon should always be investigated for punching shear in its lower portion because that, rather than arch action, is really what supports most of the load. However, it does not make any difference how a dam is held in place as long as it remains there with a sufficient factor of safety.

To overhang the crown of an arch down stream is a practical way to obtain minimum length of radii for the lower arch elements. This can be secured to a still higher degree by overhanging the dam on the up-stream side of the abutments as well. Several such dams have been built and are projected.

Such a dam, however, is not a cone dam; both the up-stream and down-stream faces are warped. Merely to incline the dam in the middle does not produce conical surfaces at any place. Fig. 5 shows plainly that the up-stream face projects a maximum at Section *B-B*, and that this projection diminishes toward the abutments where it may become zero or negative. The axis on which the centers lie is not a straight line in any plane and therefore neither face could be a cone as described by the author.

The author states that "the arches are of uniform thickness at the upper elevations, but are thickened toward the abutments, in the lower portion of the dam." That feature does not make a conical surface out of it. Both faces of this dam are warped in a way that is characteristic of the "constant angle arch" type of dam.

---

[17] Cons. Hydro-Elec. Engr., San Francisco, Calif.

In 1911, the writer gave the type of dam shown in all the illustrations this name. It seemed the best available, in order to emphasize that the important thing was to keep the subtended angle constant and large at all elevations. While the lengths of the radii are variable from crest to foundation, this is not a characteristic but an incident; this fact was the main reason for not giving the name of, say, "variable radius" arch dam.

The Railroad Canyon Dam shown on Fig. 10 and Fig. 11, appears to be of the same type and the only new feature on this design is the forked abutments.

It is very possible to construct an arch dam in a V-shaped canyon, such that the up-stream and down-stream faces would be inverted cones. Such a structure could rightly be called a cone arch dam; but it seldom would be practical. Figs. 5 and 7 have warped surfaces and are circular, or nearly circular, in horizontal planes, but they are not cone-shaped arches.

William Cain,[18] M. Am. Soc. C. E. (by letter).—The main object in the proposal of the use of the laminated arch dam, as defined by Mr. Noetzli, is to substitute a series of "thin arches", properly combined, for the "thick arch" as usually constructed. The theory of the thick horizontal arch under uniform normal loads and temperature changes, has been developed by B. F. Jakobsen, M. Am. Soc. C. E.[19] the writer, in his discussion,[20] giving an independent analysis of the problem. The application to actual structures often shows that so much tension is exerted at certain points—the extrados at the abutments and the intrados at the crown—that numerous cracks will form there, so that it is proposed to ignore this part of the primary arch that is in too much tension and to substitute for it a so-called "secondary arch". The effect is to place in the dam material that is structurally inoperative, which, Mr. Noetzli claims, is poor design.

The advantage of the laminated dam is that the corresponding theory is much simpler—approaching that of the "cylinder theory"—but it must be borne in mind that even for the thin arch, cracks are to be expected (as witness the many cracks experienced in the Stevenson Creek Experimental Dam[21]), so that the theory that supposes no cracks to form, is again not strictly applicable, even for the thin dam.

In the Stevenson Creek Experimental Dam, cracks occur at parts of the base and at the side walls of the canyon, as well as at other points; so that the dam cannot be regarded as "fixed" over such parts of the base or side walls where cracks occurred. Further, for the upper part of the dam, the arches were deflected down stream for the central portion and up stream for parts nearer the abutments, so that no existing theory applies for much of the dam.

---

[18] Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C. Professor Cain died December 6, 1930.

[19] See "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 475.

[20] *Loc. cit.*, p. 522.

[21] *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3.

In the theory best known, the dam is considered artificially divided into a series of horizontal arches and vertical cantilevers and the shear between them is neglected, whereas the true solution would be to consider the arched dam as a whole, including the shear, to which the strict theory of elasticity should be applied for a solution. When this is done, doubtless many surprises are to be looked for, because the results will hardly agree with present approximate solutions, especially if the analysis, as usual, proceeds on the hypothesis of no cracks or the dam remaining intact.

The trial-load method has been applied, assuming the usual artificial division of the dam into arches and cantilevers, ignoring the shear between them, and, in addition, regarding as operative only those parts of the sections on which no tension is exerted. This necessarily leads to inexact results, for plain concrete can take a certain amount of tension—possibly 100 lb. per sq. in. or less. The result would also be inexact if the dam is supposed to remain intact and thus to be able to resist whatever tension would be exerted.

On all these accounts, the problem is seen to be very complex, so that any device that would lead to a simpler approximate solution, should prove welcome. This has been attempted by Mr. Noetzli in his so-called laminated dam, built up by a series of thin dams, to which the theory of thin arches applies, so that the solution, as the thickness of the arch diminishes, approaches that of the cylinder theory. Another great advantage is that the stiff, thick cantilevers of the thick arch, are replaced by thin, flexible cantilevers, which carry very little load, most of it being transferred to the horizontal arches. On that account, possibly the very laborious trial-load method can be omitted and a uniform normal load on the horizontal arches assumed, for which the theory is comparatively simple.

The author further proposes the use of forked abutments, which should give a very stiff structure and thus ensure that the arches are practically "fixed at the ends", and that shorter spans and more favorable radii will result. There are also other advantages, to which the author calls attention, in the proposed sloping of the crown of the arch. It is a question likewise as to whether the arch dam should not be constructed with vertical radial joints, at intervals, to be subsequently grouted at low temperatures, and also whether the cantilevers should not be simply supported (or hinged) at the base, so as to throw as much of the load as possible on the arches. The writer has given the solution for the latter case as applicable to the Wooling Dam,[22] using the method[23] of B. A. Smith, M. Am. Soc. C. E.

The formulas and diagrams for the computation of the stresses in the thin horizontal arches are based on the writer's theory for the thin arch.[24] For example, the total tangential stress at the point (r, ϕ) is:[25]

$$P = p\,r - (p\,r - P_0)\cos\phi = p\,r - R_r\cos\phi \quad \text{.........} \quad (9)$$

---

[22] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 311.

[23] *Loc. cit.*, Vol. LXXXIII (1919–20), p. 2027.

[24] "The Circular Arch Under Normal Loads," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

[25] *Loc. cit.*, p. 236, Equation (4).

Dividing Equation (9) by 144 t, the result is the total rib-shortening stress, in pounds per square inch. In this equation, the first term, $\frac{p\,r}{144\,t} = \frac{p'\,R_u}{144\,t}$, gives the cylinder stress, σ_c, of the author's Equation (2); the second term, $-\frac{H_r \cos\phi}{144\,t}$, gives the stress due to the pull, H_r, at the crown, tending to lengthen the neutral axis. The algebraic sum gives the total rib-shortening stress.

Assume, for convenience, that $y = \left(\frac{\sin\phi_1}{\phi_1} - \cos\phi\right)$; then the moment at (r, ϕ) is:[24]

$$M = R_r\,r\,y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(10)}$$

so that the stress due to this moment at the intrados or the extrados, in pounds per square inch, is:

$$\frac{1}{144}\,\frac{6\,M}{t^2} = \frac{1}{24}\,\frac{H_r\,r\,y}{t^2} \ldots\ldots\ldots\ldots\ldots\ldots \text{(11)}$$

Hence, the stress at intrados or extrados, due to the pull, H_r, and the moment is,

$$\sigma_r = -\frac{H_r\cos\phi}{144\,t} \pm \frac{H_r\,y\,r}{24\,t^2}$$

This is the author's Equation (5),[26] in which $\phi = 0$ at the crown and $\phi = \phi_1$ at the abutment. The total unit stress, is thus, $\sigma = \sigma_r + \sigma_c$, as in the author's Equation (8).

Similarly, the results for temperature changes are derived for the free arch, not including the effect of the restraint due to cantilever action. The formula given for σ_r is not the rib-shortening stress, which is strictly,

$$\sigma = \frac{H_r\cos\phi}{144\,t} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(12)}$$

and is uniform over the section of thickness, t.

The author gives in Fig. 9, a plan of a French dam consisting of five, thin, reinforced arches, placed one below the other, 60 ft. or more apart, the heights decreasing down stream. The principle of lamination is well illustrated in the design which has many advantages and some disadvantages. As to the latter, the extra form work is objectionable, and possibly the arches are not fixed at the abutments so well as in the author's design. His laminated arch is very interesting, and it is to be hoped that it will be fully tested in an actual construction.

B. F. JAKOBSEN,[27] M. AM. SOC. C. E. (by letter).—The "laminated cone arch dam" appears to be a constant-angle arch dam,[28] provided with one or more asphalt-covered slip-joints, which are concentric with the up-stream face of the dam. These joints are assumed to permit minute movements and adjustments between the separate arch sections. The author is so certain that this must happen, that he suggests a test of the factor of safety be made by loading each arch lamina of a model separately. That process amounts to taking for granted just what is most in need of proof, namely, whether or not these slip-joints, the areas of which must be measured in acres, will function as assumed. A test on a model would give no reliable information as to whether or not several acres of asphalt-painted surfaces will slide more or less freely on each other; furthermore, it would give no information as to how aging, temperature variations, etc., would affect the coefficient of friction. The proposed design has no application to small structures, since there is no decided advantage in laminating the arch unless it is quite thick. For comparison, it may be stated that the up-stream face of the proposed San Gabriel Dam of the Los Angeles County Flood Control District, has a surface area of more than 20 acres.

---

[26] "The Circular Arch Under Normal Loads," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 237, Equation (13).

[27] Cons. Engr., Los Angeles, Calif.

[28] "The Constant-Angle Arch Dam," by L. R. Jorgensen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 685.

The author refers to the successful testing of the Stevenson Creek Arch Dam and suggests that a simple test on a small model would furnish the factor of safety of his proposed design. A comparison of the design data for the Stevenson Creek Dam with the test data, and the values obtained from the celluloid model, suggests caution rather than confidence.[29] There appear to be too many factors which the model may not reproduce correctly, but which would affect the safety of the structure.

Mr. Noetzli assumes a coefficient of friction of 0.5 for the asphalt-painted joints. This may be a fair assumption under ideal conditions, for small areas, while the asphalt is fresh and its temperature not too low; but such ideal conditions, involving practically true surfaces, as in a machined joint, will not be obtainable in large dams. Furthermore, the effect of shrinkage, swelling, temperature variations, etc., will warp these surfaces, and the total result is likely to be high compression between the two surfaces at various points and no compression at other points. This will tend to increase the frictional resistance.

The friction here involved is that of rest following a considerable period of no movement, during which time the surfaces have been pressed tightly together, and the movements are minute, as stated by the author. These are the conditions most unfavorable for a small coefficient of friction.

The writer is not able to follow the author's calculations of the influence of friction, but if any friction whatever exists, it will defeat the purpose of the lamination to some extent. It is also clear that there will be no movement at all, where the shear is less than the product of compression between the surfaces and the coefficient of friction. That the average shear for the whole surface is greater than the average frictional resistance, will hardly suffice, because relatively small movements at places where there is an excess of shear will materially decrease that shear. Such movements will not greatly affect the frictional resistance, because the compression between the surfaces will not be altered materially. At the crown section the shear is zero and the compression is considerable; there is no need for motion between the surfaces right at the crown section; but immediately outside, a relative movement is required and the shear is still very small.

---

[29] *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1241.

As an actual example the writer has considered an arch with the ratio, $\frac{t}{r} = 0.5$, and the central angle, $2\,\phi_1 = 120$ degrees. Fig. 12(*a*) shows the radial compression, σ_r, at the crown section, where the shear is zero, Fig. 12(*b*) shows the radial compression and the shear stress, τ, for the section determined by $\phi = 30°$, and Fig. 12(*c*) shows the same for $\phi = 60$ degrees. These values were calculated from the writer's paper, "Stresses in Thick Arches of Dams",[30] on the assumptions that Poisson's ratio may be neglected; that is, that $m = \infty$; that the arch carries a load of 200 ft. of water, or 87 lb. per sq. in.; and that shrinkage, swelling, temperature variations, and the yielding of the abutments are negligible. The effects of Poisson's ratio, yielding of the abutments, and swelling, are probably to increase the radial compression and to decrease the shear, while the effect of shrinkage is the reverse.

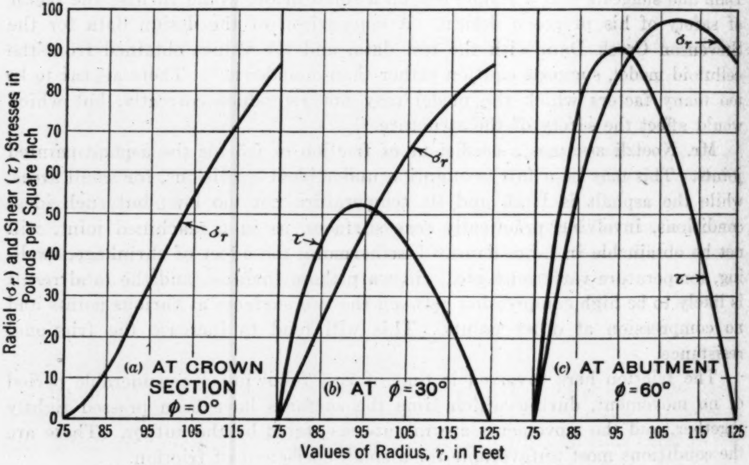


FIG. 12.—RADIAL AND SHEAR STRESSES IN AN ARCH 50 FEET THICK.

An arch 50 ft. thick would presumably be divided into three laminæ. Fig. 13 shows the values of σ_r, the radial compression, and of τ, the tangential shear, at points 16.5 ft. from the up-stream face of this arch, and Fig. 14 shows the ratio of $\frac{\tau}{\sigma_r}$. Let f equal the coefficient of friction; then no motion will result if $f \geqq \frac{\tau}{\sigma_r}$.

For values of $f < \frac{\tau}{\sigma_r}$, motion will result (for the moment, disregarding the stresses in the cantilever), and the stresses will be partly re-adjusted. The re-adjustment will be complete only if $f = 0$. Fig. 14 shows that, for $f = 0.5$

---

[30] *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 475.

(as assumed by the author), $f \geqq \frac{\tau}{\sigma_r}$ from the crown section, where $\phi = 0$, to $\phi = 28$ degrees. For angles greater than $\phi = 28°$, motion and a slight re-adjustment of stresses should result. This re-adjustment will be adversely affected by the half of the arch where the shear is insufficient to produce any movement. The average value of $\frac{\tau}{\sigma_r}$ for the whole arch is about 0.483, which is less than *f*. For these reasons the writer can not accept the author's conclusion, that the friction in the joint is negligible. On the contrary, it seems, if not evident, at least highly probable, that the friction is sufficient to prevent all but very minor adjustments of the stresses, and it would also appear that this is the safe and conservative assumption.

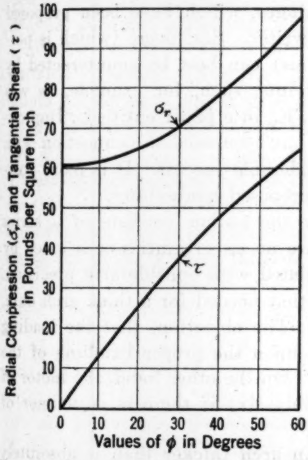


FIG. 13.—RADIAL AND SHEAR STRESSES, 16.5 FEET FROM THE UP-STREAM FACE IN AN ARCH 50 FEET THICK.

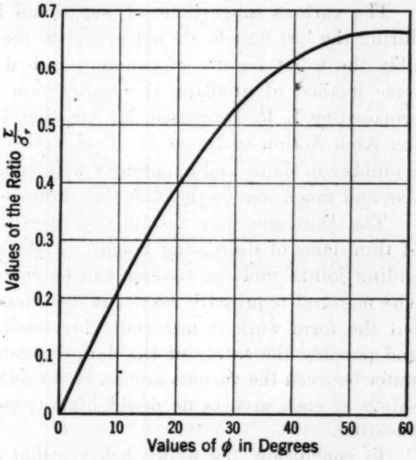


FIG. 14.—RATIO OF $\frac{\tau}{\sigma_r}$ AS DETERMINED FROM FIG. 13.

Mr. Noetzli appears to have overlooked the effect of the cantilever. The radial compression at any point in the dam is the algebraic sum of the compression due to the arch and that due to the cantilever, while the tendency to produce motion is the geometric sum of the arch and cantilever shears. As an illustration, assume arbitrarily that at any point of the arch the shear is τ and that the arch radial compression is $2\,\tau$. Then, if $f < 0.5$, motion will result. If, at this same point, the cantilever shear is also τ and the cantilever radial compression is $2\,\tau$, the incipient motion is in a direction making an angle of 45° with the horizontal; but the resulting radial compression is now $4\,\tau$, while the shear force tending to produce motion is $\sqrt{2}\ \tau$; their ratio is $\frac{\sqrt{2}}{4} = 0.354$ and, consequently, no motion will result, unless f is less than 0.354. The influence of the cantilever is, therefore, a distinct disadvantage and it will tend to prevent motion.

The object of the theory of the secondary arch is not to determine the actual stress distribution, but to establish a limiting maximum value of the stresses, as the writer has pointed out.[31] The author is quite correct in stating that a thick arch involves inherent complexities and uncertainties; but these, the writer believes, are insignificant compared to the complexities and uncertainties involved in determining the degree of stress adjustment brought about by the laminations, because this involves everything that concerns a thick arch and a thick cantilever, with the added complications of small local movements and local stresses at the joints and the uncertainty of the coefficient of friction. These laminations, like the devices for underdraining a gravity dam, may work to some extent, and they may not; the safe assumption is that they may not function.

The various suggestions of joints and hinges, which have been proposed during the last decade, do not appeal to the writer. Shrinkage (which is possibly the worst feature of concrete arch dams) can best be counteracted by some method of grouting the contraction joints, such, for example, as was proposed by L. R. Jorgensen, M. Am. Soc. C. E., in a paper entitled, "Improving Arch Action in Dams".[32] This method has been used in connection with a number of dams and apparently with considerable success. It is more positive and much less costly than the author's proposed laminations.

The Dordogne River Dam described by the author consists of a series of thin dams of decreasing height. There are no uncertainties occasioned by sliding joints, and the stresses can be calculated with considerable precision. The material required is no doubt less than that needed for a thick arch dam, but the form work is materially increased. The objection, that the loading and possibly the safety of the dam depends upon the proper handling of the water between the various arches, is obvious. On the other hand, the factor of safety of each arch is no doubt high (possibly six) if there is no danger of buckling.

In conclusion, the writer believes that an arch thicker than is absolutely required is not an unmitigated evil nor necessarily a mere waste. The outside concrete protects the interior load-carrying portion from weathering, temperature fluctuations, and rapid drying out. In addition, if a thick arch is stressed beyond the elastic limit of the concrete at any place, a redistribution of stresses will take place, as discussed by the writer,[33] and the thick arch will be found to have a materially higher load-carrying capacity than ordinary theory indicates.

P. WILHELM WERNER,[34] ASSOC. M. AM. SOC. C. E. (by letter).—The author aims at a rational use of the arch principle through application of arch lamination and of forked abutments. The views expressed in the paper are of much interest to the writer because of certain investigations made along the same lines.

---

[31] *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), pp. 510 and 576.

[32] *Loc. cit.*, Vol. LXXXIII (1919–20), p. 316.

[33] *Loc. cit.*, Vol. 90 (June, 1927), p. 507, Article XVII, "Stress Distribution at Failure."

[34] A. B. Vattenbyggnadsbyran, Stockholm, Sweden.

Concerning the arch lamination, the writer believes that the design, from a theoretical point of view, is a step toward the perfect arch dam. As laid out by Mr. Noetzli, the construction is also practicable and economically feasible.

As regards the forked abutments, the main question seems to be to provide for monolithic action of the abutments. In general, it is probably possible to insure safety of the abutment in regard to sliding, overturning, and other rules governing the design of a gravity section, provided the abutment could be assumed to act as a monolithic structure. In the writer's opinion it is questionable, however, whether it is possible to accomplish this with either the forked abutment or the so-called gravity tangent.

Referring to Fig. 5, the abutment of Dam No. 1 has a length of about 250 ft. and a height of about 75 ft. It is generally conceded that a straight gravity concrete structure with these dimensions would inevitably show transverse cracks in the masonry, unless contraction joints are provided at suitable intervals. However, if cracks should occur, or if the transverse joints should open up, it is easily conceivable that the abutment would not be fit to transmit the thrust to the side-hills. Even if pressure groutings are applied, it does not seem to be an ideal way of finishing an arch dam. Unless special precautions are taken to insure monolithic action of the abutments, or unless the conditions at the site are specially favorable for their construction, the writer does not feel inclined to rely upon either the forked abutment or the gravity tangent for transmitting the arch thrust to the rock. This point should receive careful consideration. It would be interesting to know how the problem has been solved in the cases referred to by the author.

H. B. MUCKLESTON,[35] M. AM. SOC. C. E. (by letter).—Two planks side by side will carry twice as much load as one plank. If one is laid over the other, with rollers between, the combination will still carry twice as much. If the two are in contact, the combination will carry rather more than twice and, if the two are clamped together, the carrying capacity will increase until, when the friction between them exceeds the longitudinal shear in a simple beam of the same depth, the combination will act as a simple beam and carry four times the load.

The author's expedient of laminating an arch dam in order to avoid the disadvantages of the thick arch will be successful according as he can avoid the frictional resistance between the laminations. This he proposes to do either by lubricating the surfaces by creating a film of asphalt, or by a physical separation of the surfaces, using hydrostatic pressure to transfer the proportionate part of the load from one lamination to the next.

The first method would be excellent provided the surfaces were truly circular in horizontal section with no departure from the true circle exceeding one-quarter, or thereabouts, of the thickness of the lubricating film. It is more than doubtful whether a large dam could be built with no departure greater than ½ in., which means that the asphalt film would be at least 2 in. thick if the benefit is to be realized. This is many times the thickness of the capillary film, even for asphalt, and, as a consequence, the film would exert hydrostatic pressure equal to its full height (even when it appears to be brittle, asphalt is a highly viscous liquid). This would act to transfer the proportional part of the load when the reservoir was full; but when the reservoir was empty, it would press the up-stream lamina in a radially up-stream direction with a greater tangential tension as a result than the material could stand, unless it were heavily reinforced for the special purpose.

---

[35] Cons. Engr., Vancouver, B. C., Canada.

The second method, by a physical separation, using water as a medium to transfer the load, would work when the reservoir was full; but when the reservoir was empty, the space between the laminæ must be emptied also, and this means manual or automatic control. In either case, such factors as silt, débris, ice, and deliberate human intervention—which is by no means impossible—offer too many possibilities of trouble to make the idea comfortable. The thought of deliberate human intervention may seem like borrowing trouble, but it must be remembered that the "cranks" and anarchists are not all dead by any means. It is not long since that one of them tried to wreck the Welland Canal for apparently no good reason, while, during the World War, enemy sympathizers on more than one occasion undertook the deliberate destruction of bridges and other works in Canada.

In one respect, the paper is to be commended. Mr. Noetzli seems to have avoided successfully the idea of wasting acres of paper on the vain attempt to divide the load between the arch and cantilever elements to nine places of decimals. This extreme exactness has always appealed to the writer as an attempt to split a mathematical hair, with the purpose of adding the weight of the left half to that of some saw-log the weight of which has to be guessed anyway.

Fredrik Vogt,[36] Assoc. M. Am. Soc. C. E. (by letter).—This interesting paper describes three different features in the design of arch dams, *viz.*, the forked abutments, the overhanging type of "constant-angle arch dam," and the laminated arch elements.

The construction of numerous dams, for instance, Salmon Creek Dam designed by L. R. Jorgensen, M. Am. Soc. C. E., the Kerckhoff Dam designed by B. F. Jakobsen, M. Am. Soc. C. E., and the Gibson and the Deadwood Dams designed by the U. S. Bureau of Reclamation, has proved that single-span arch dams can be used where the sites are about as favorable or less favorable than those shown in Figs. 5 and 7, and without using high gravity wings. When the arches can be founded directly on the rock without high wings or buttresses, this solution seems to offer many advantages. In some cases, however, the topography of the site may invite the use of forked abutments. The Stewart Mountain Dam, of the Salt River Water Users' Association, is an example of forked buttress and gravity wings. In the analysis of such dams it should not be forgotten that the tangential compression of the buttress will increase the span of the arch and thus will increase the rib-shortening pull and the bending stresses.

---

[36] Docent, Norges Tekniske Hölskole, Trondhjem, Norway.

There is no doubt that the thrust in the arches is inclined at the abutments as assumed by the author, particularly at sites that require a large ratio of span to height. The writer has previously discussed this matter,[37] and the theoretical analysis given at that time was later checked by model tests. Owing to this inclination low buttresses, one-eighth to one-fifth of the height of the dam, will not receive much load if used at sites as shown in Figs. 5 and 7. It is a question whether gravity wings should not be limited to such heights, where possible. For instance, in the case of Dam No. 1 shown in Fig. 5, arch elements with the same radius as that used in the design, but with greater central angles, could be used for greater spans, and thus the arch elements could be founded directly on the rock for 75 to 80% of the total height of the dam instead of only 53% as noted in the design, thus reducing the wings considerably. Such a change will give a smaller rib-shortening pull and smaller stresses in the arch elements; it will give practically the same stiffness against deflection[38] and also practically the same cantilever stresses. In other words, the efficiency of the arch elements will be improved in some respects and, in others, will be virtually unchanged.

However, this possibility of increasing the span of the arch and thus reducing the wings without increasing the radius is due to the fact that rather small central angles of about 90°, have been used at the crest for both Dams Nos. 1 and 2.

As shown by Mr. Jorgensen,[39] it is preferable to use central angles of about 120°, assuming that the cylinder formula is valid. If analyzed by the now generally accepted elastic theory, including also temperature changes, central angles of 150 to 160° are found to give the minimum of concrete, and, therefore, should be preferred where it is possible to obtain good abutments with such large angles.

In the designs of Dams Nos. 1 and 2 the central angles applied at the base of the buttresses are about 130° and 120°, respectively; that is, they are about as suggested by Mr. Jorgensen. Higher up the span of the arch is unchanged, and it is difficult, therefore, to find any reason for the increase of the radius of the arches and the accompanying reduction to 90° at the crest in the central angles. The latter only reduces the efficiency of the arch elements. The writer has emphasized[40] the fact that this use of a variable radius is in disagreement with the constant-angle arch principle and is a disadvantage. If the intention is merely to obtain overhanging arches, this result could easily be secured by the use of a constant radius and a constant angle for the height of the buttresses. For both designs the direction of the buttresses permits central angles as large as 130 degrees.

The first "constant-angle arch dams" were constructed without appreciable over-hang, but where larger angles can be obtained by this means, an overhang seems to be well justified. Previous to the Railroad Canyon Dam referred to by the author, such overhanging dams had been constructed in Italy

---

[37] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1272.

[38] That is, practically the same coefficient, λ, in the deflection formula, as shown in *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 564, and Fig. 46, p. 565.

[39] "The Constant-Angle Arch Dam," by Lars R. Jorgensen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 685.

[40] *Engineering News-Record*, 1928, p. 707.

(Chiusella Dam, 158 ft. high, completed in 1925[41]); and in Mexico (Calles Dam, 208 ft. high, completed in 1928[42]). If such over-hang is not to result in vertical tension in the cantilevers, pressure grouting must be applied in the radial contraction joints.

The Dordogne River Dam, in France, referred to by the author in Fig. 9, typifies the spirit of many French engineers. The stresses in the Dordogne River Dam are as much as 800 lb. per sq. in., due to water load only, to which must be added the stresses due to the shrinkage and temperature changes. In this connection reference should be made also to the Selune and Belle Isle en Terre Dams, in France. In all the modern arch dams in France[43] the thickness of the arches is reduced to the limit—for the two last-mentioned dams to 7 and 7½ in. at 50 and 54 ft. head of water—by using very short spans (about 16 ft.) between the buttresses. The experience with dams in the United States, and in Scandinavia, seems to show that this scrimping in thickness, down to the theoretical limit, is not advisable, at least not in cold climates.

In any case, the system applied in the Dordogne River Dam is clear in its action, and the maximum stresses can never be very much greater than the cylinder stresses. However, the safety of such dams is controlled either by automatic valves, or by hand operations of the dam keeper. If the basins between the arches are not filled as water in the main reservoir is rising (for instance, because the valve openings are clogged by ice), or if for any reason they are emptied before the main reservoir is emptied (for example, by a great leakage or a bursted valve or pipe), the maximum stresses can become more than 4 000 lb. per sq. in., and the dam is in danger of collapsing. Furthermore, if the reservoir is emptied and the basins are not, the dam will fail. The writer has seen an earth dam which failed because it was dependent on a spillway with an automatically moving gate and because the dam keeper did not maintain that gate in good working order during the cold season. After observing such field practices, he is suspicious of types of dams which depend for safety on factors that may not be proof aginst erroneous operation. Dams ought to be constructed with higher safety factors than most other structures.

The site of the Dordogne River Dam would be excellent for a single-span arch dam, with a span at the crest only 1.8 to 1.9 times the height, which is less than for two-thirds that of American dams higher than 100 ft. The construction cost of a structure of the accepted type scattered over a wide area and with greatly increased excavation and form work, will probably be considerably more than for a single-span arch dam, even if the latter should demand more concrete when designed for the same stresses.

Compared with the French dam, the lamination suggested by the author has the advantage that the action is independent of automatic devices. On the other hand the static action is not so clear. The author states:

"Assume an elementary horizontal arch with two arch laminæ at a depth of 200 ft. below the crest of a dam. The water pressure at this depth is 62.5 × 200 = 12 500 lb. per sq. ft. The up-stream lamina is assumed to carry one-half this pressure, the other half (6 250 lb. per sq. ft.) being transmitted across the joint to the down-stream lamina. Assuming a coefficient of friction in the asphalted joint of 0.5, the frictional resistance in the joint, therefore, would be 0.5 × 6 250 = 3 125 lb. per sq. ft. (22 lb. per sq. in.). This is rather small as compared to the difference in stress in the concrete on opposite sides of the joint, which may be as much as 200 to 300 lb. per sq. in. It is evident, therefore, that the friction in the asphalted joint between the arch laminæ is not likely to restrain them sufficiently to alter the stresses materially from those indicated by theory."

---

[41] *Transactions*, First World Power Conference, Lond., 1924, Vol. II, p. 591.

[42] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1310.

[43] No more than these three are known.

It should be remembered that the frictional forces involved must be added from the crown section to the abutment, and that, therefore, the total frictional force on each half of the arch will be 50% of the water load on the same half transmitted to the lower lamina if the coefficient of friction is 0.5. If frictional forces 0.5 times the water load are acting in the joint between the two laminæ in the arch shown in Fig. 8 for which the stresses are given in Table 2, the writer has computed additional stresses at some places as great as 680 lb. per sq. in. for that arch according to the elastic theory. This added to the stresses given in Table 2 would increase the maximum total stresses by 110%, which is far beyond all reasonable limits.

However, the actual result will not be so bad since the friction will redistribute the division of the water load on the two laminæ, and the laminated arch, therefore, will not be stressed more than the assumed homogeneous solid arch, but will act about the same.

Another and better way to get an estimate of the importance of the friction in the joints is to investigate the stresses at the assumed joints as they would be if the arch element was made solid. If the shear stresses in the solid arch, acting along the surface where the joint would be, are smaller than the radial compression acting normal to the same surface, multiplied by the coefficient of friction, no sliding can be expected, and the laminated arch will act as a solid arch. Mr. Jakobsen has analyzed such stresses in thick arches.[44] With the simplifications which can be used for slender arches the writer has computed these stresses for the arch element shown in Fig. 8, and finds at the surface of the suggested joint the stresses due to the water load, as follows:

Shear Stresses:

At the crown section,

$$\tau = 0$$

at the abutment section,

$$\tau = 0.0411\,\sigma_0$$

the average for the total length,

$$\tau = 0.0228\,\sigma_0$$

Radial Stresses Normal to the Surface:

At the crown section,

$$\sigma_r = 0.0452\,\sigma_0$$

at the abutment section,

$$\sigma_r = 0.0753\,\sigma_0$$

the average for the total length,

$$\sigma_r = 0.0533\,\sigma_0$$

---

[44] *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 503; see, particularly, Fig. 17, p. 505.

in which, the values of σ_0 are the stresses according to the cylinder formula. With a coefficient, $f = \frac{\tau}{\sigma_r}$, the friction will prevent all movements and will make the arch act as a solid one. In this way the limit of the coefficient of friction is found to be 0.00 at the crown section, $\frac{0.0411}{0.0753} = 0.545$ at the abutment section, and an average for the total length of $\frac{0.0228}{0.0533} = 0.427$.

For very thick arches this limit approaches unity; for instance, for $t = 0.5r$ and $2\phi_1 = 90°$, the limit is 1.0 at the abutment and about 0.6 as an average.

If the actual coefficient of friction is greater than these limits, the arch will act as if it were solid. If it is smaller, some sliding will be started. However, a sliding will reduce the shear stresses considerably without changing the radial stresses very much. The sliding, therefore, will stop as soon as the shear stresses are reduced to equivalence with the frictional forces. In Table 2 the author has found that for this particular arch the lamination creates a 19% reduction in stresses if there is no friction. With a coefficient of friction of 0.5, used by the author, the actual stress reduction is obviously zero, or nearly so. If a coefficient of friction equal to 0.2 could be obtained, a stress reduction of about 10% could probably be depended on, since the average limit corresponding to no reduction is 0.43.

In comparing the stresses it should also be remembered that the yielding of the foundation reduces the maximum compression in the arch, and that the percentage reduction is greater for thick arches than for thinner or for laminated ones.[45] This correction is not included in Table 2, which, therefore, is not quite fair for the solid arch.

Asphalt can be obtained in a number of grades by admixing with oil; all these grades are liquid if warmed, stiffer if cooled, and brittle and hard if sufficiently cold. Fig. 15 shows how asphalt used as a seal in a water-stop of a dam, has been acting as a liquid. The square opening for the asphalt seal was placed too close to the face of the dam, and the pressure from the semi-liquid asphalt broke the corner between the seal and the face. The writer was informed that this happened because of the summer heat and not by reason of artificial heating. The pressure was developed too fast to be released by slow leakages. This does not mean that the same asphalt could not run through narrow openings. At another joint the writer saw asphalt which had flowed 40 ft. from the up-stream face, through a contraction joint with an 0.03-in. opening at the down-stream face. For a part of the way it may have followed the grouting pipes, but the openings to and from these pipes are generally not wider than the opening of the joint, and at the down-stream face the asphalt was coming out of the joint and not through the pipes (see Fig. 16). The same asphalt was brittle and hard in winter.

---

[45] Compare the writer's discussion on "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 554, particularly Tables 11 and 12. Later, the basic formula is checked by the Stevenson Creek Test Dam results, see *Proceedings*, Am. Soc. C. E., Pt. 3, May, 1928, pp. 124–129.

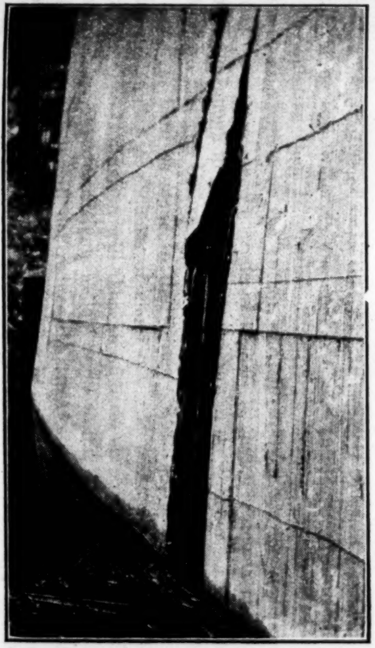


Fig. 15.—Blowout Due to Static Pressure from Semi-Liquid Asphalt.

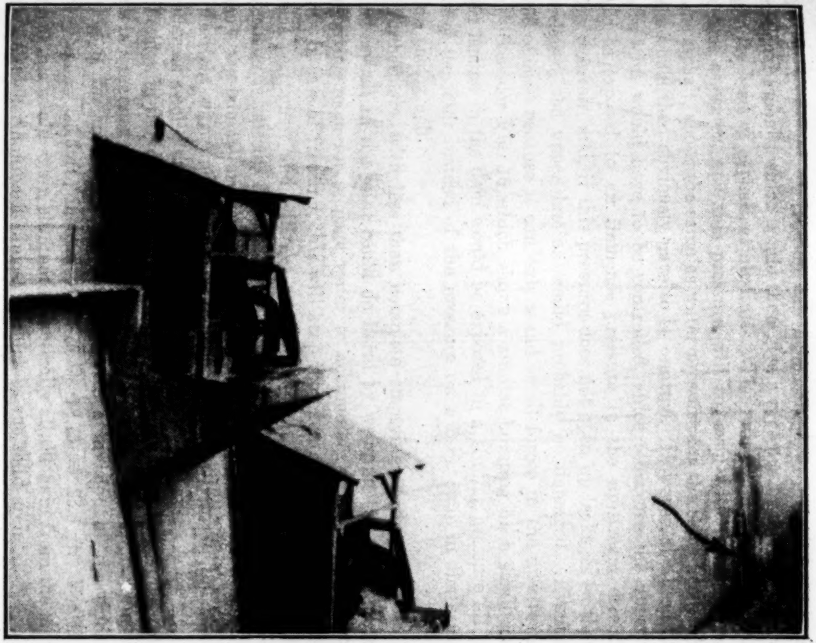


Fig. 16.—Arrow Shows Place at Which Asphalt Was Leaking Out at Down-Stream Face from Water-Stop at Up-Stream Face and Through Narrow Contraction Joint.

A low coefficient of friction can be obtained by using a thick layer of semi-liquid asphalt, which would then give a pressure in the joint corresponding to the head of asphalt at all times. The unit weight of asphalt is about the same as water, and this pressure will be, even with rather slow-flowing plastic asphalt, about the same as water pressure for the same head. All open drains would then obviously have to be omitted. However, the head of liquid in the joint would have to be controlled, since otherwise the upper lamina would be exposed to an immense pressure in the up-stream direction for empty reservoir, and, if this pressure does not open up leakages so that it can be released, the upper lamina would probably be damaged. Eventually, it would collapse because it can only stand small forces in the up-stream direction. Consider Fig. 15 which shows a similar bursting on a small scale; the upper lamina in the dam would be exposed to a similar pressure all over the area of the joint instead of the pressure on a 5-in. strip at the asphalt-seal in the photograph.

Furthermore, with less than complete emptying of the reservoir a semi-liquid asphalt in the joint could do harm by wedging the two laminæ apart, thus giving room for seepage water to accumulate in the upper part of the joint. In case of freezing, this will keep the opening at the crest of the lower lamina permanent and will act as a permanent ice wedge also in case the water later is raised. In this way great extra forces can be introduced both because the laminæ are wedged apart and, therefore, cannot divide the load in the right way, and because a freezing action will make the laminæ act together as a solid arch.

Besides, in the ideal case, the use of a semi-liquid asphalt would divide the loads in a manner similar to that for the Dordogne River Dam. For the arch in Fig. 8, for instance, this would require a division of the thickness in $20.4 + 5.1 = 25.5$ ft., instead of $12.75 + 12.75 = 25.5$ ft. For that arch element this would again eliminate practically all the gain from the lamination since the thicker lamina is almost as thick as the solid arch.

The author does not seem to plan on the use of any liquid or semi-liquid asphalt, and this discussion is only inserted to show that, if one tries to obtain a low coefficient of friction in that way, it is better to adopt a system of controlled filling with water as was done with the Dordogne River Dam.

The question then is whether coefficients of friction less than 0.5, for instance, as assumed by the author, can be obtained for the asphalted joint during the winter, when a grade of asphalt is chosen which is not liquid or semi-liquid enough to produce static pressure in the summer. The writer does not believe that can be expected; the fact is that such grades of asphalt will be almost as hard as stone in the winter. With surfaces made smooth by using steel forms, coefficients of about 0.5, and perhaps as high as 0.7, are probable. The lamination will then be of little or no use for equalizing the arch stresses. With coefficients as low as 0.1 to 0.2, the lamination will be of some use even if it will not reduce the stresses as much as the author has assumed, unless the coefficient of friction is zero.

The author has computed the relative movements at the joint, using a modulus of elasticity of 3 000 000 lb. per sq. in. According to the tests made by Raymond E. Davis,[46] M. Am. Soc. C. E., it does not seem fair to assume more than 1 500 000 lb. per sq. in. in case of long-time loadings, which will double the relative movements. If the friction is zero, the vertical sliding at some places will usually be several times as great as the horizontal sliding. Friction against this will reduce the effect of the lamination on the cantilever stresses in the same manner as it reduces the effect on the arch stresses.

The lamination of the arch will make this less homogeneous than before. The asphalted joint will act as a water-stop in the middle of the dam, the effect of which will extend all over the structure and, in all probability, it will keep the upper lamina wet from seepage water and the lower lamina dry, since the seepage water is stopped at the joint and drained off there. This at first will produce a considerable shrinkage in the lower lamina and little or no shrinkage in the upper lamina. Secondly, it will give a "sustained modulus of elasticity" for continuous load on the dry lower lamina, which is only about one-half that for the wet upper lamina.[47] The difference in modulus of elasticity in the ratio of 1:2 will change the division of load from 50 + 50% to 33 + 67%, and thus will give additional stresses of 33% for the upper lamina. The difference in shrinkage will exaggerate this condition considerably; a difference of 0.02%, for instance, will introduce another 150 lb. per sq. in., equal to 25% additional stress if based on 600 lb. per sq. in.

It is true that there will be some effects of this kind also in solid arches, but without any doubt the use of a water-tight membrane in the middle of the dam, dividing this into an upper wet and a lower dry part, will increase the non-homogeneity, and thus will increase such additional stresses materially. In a solid arch the water-soaking will be rather uniformly distributed due to capillarity and water pressure; only at the down-stream face will the concrete be dry.

If there is no friction in the joints, the cantilever elements would become comparatively ineffectual, as stated by the author, and the cantilever stresses would be reduced. However, it should be remembered that this is not always desirable. For instance, for the Gibson Dam,[48] the arch stresses would no doubt be much too high if the cantilever action were made ineffectual, and this, therefore, would demand a heavier arch section.

Conclusion.—The foregoing discussion leads directly to five conclusions, as follows:

(1) With a coefficient of friction of 0.5, as assumed by the author, the friction will eliminate the benefit of the lamination either entirely or to a very great extent.

(2) The friction can be reduced by using liquid or semi-liquid asphalt grades, but if a liquid is used in the joints, even if it is rather viscous, the filling must be controlled, and then the Dordogne River Dam with water between the arches would prove to be a better pattern.

---

[46] *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 213; and *Transactions*, Am. Soc. for Testing Materials, 1929, Pt. II, p. 678.

[47] See tests by Professor Davis, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 213; and particularly *Transactions*, Am. Soc. for Testing Materials, 1929, Pt. II, Fig. 19.

[48] Constructed by U. S. Bureau of Reclamation, Div. of Load, see *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1295.

(3) The asphalted joint will make the concrete non-homogeneous (different moisture content in the laminæ), and this will introduce additional stresses which are much greater than the reduction which could be obtained by lamination in the case of no friction.

(4) The writer believes that, in most cases, it is desirable either to avoid gravity wings and buttresses or to reduce their height by using a longer span for the arch than is shown in Figs. 5 and 7. The most favorable height of gravity wings can only be found, of course, by comparative designs.

(5) The use of over-hang of the crown cantilever is believed to be a great advantage where the constant-angle arch principle can be better accomplished in that way.

PAUL BAUMAN,[49] M. AM. SOC. C. E. (by letter).—The author has endeavored to eliminate complexities in the interior stability of thick concrete arches by designing laminations to furnish a simple substitute for the cumbersome trial-load method. Unfortunately, his new type which, in principle, is a "telescoped" Dordogne Dam, involves serious physical uncertainties which might readily impair the safety of the structure.

The assumption, for example, that the flexibility of the compound structure is governed by that of the individual rings is hardly justifiable as long as the faces of the rings are in contact. The relative motion of molecules (relative lengthening and shortening) at contact points would add to the restraining influence of sliding friction so as to render the effective frictional coefficient very uncertain.

Furthermore, this coefficient is not a constant for one and the same point; it is a maximum at the time that movement begins (end of rest), and diminishes as movement continues. This feature would tend to cause slips between the laminæ, resulting in "dam quakes", just as it produces slips between geological strata resulting in earthquakes. The ensuing vibrations would not be desirable, to say the least.

On the other hand, it cannot be denied that the lamination would enhance the flexibility of the compound structure and, therefore, would tend to reduce the eccentricity of the line of pressure. As the load on a dam, under normal conditions, is gradually increasing during the initial filling of the reservoir, the up-stream ring would be engaged first; then the next ring would be engaged, etc.

If the spacing between the down-stream face of the up-stream ring and the up-stream face of the one adjacent to it were such as to cause uniform contact as soon as the up-stream ring has its share of the full load on the dam, its elastic behavior, up to this point, would undoubtedly be independent of the remainder of the laminæ. Based on the same premises, a further increase of the load would then engage the first plus the second ring, then the first plus the second plus the third, etc. The nth lamina would then be engaged for a load equal to $\frac{n-1}{n}$ times the total load. As soon as contact takes place between two faces, relative motion could undoubtedly still develop, but, even if such motion were disregarded, it is evident that the eccentricity of the line of pressure—and, consequently, the extreme fiber stresses of n combined laminæ—would be materially smaller than that of a monolithic dam of equal cross-section. The structural difficulties involved in such a spacing of faces to provide for deflection would be very serious indeed.

---

[49] Chf. Designer, Quinton, Code & Hill—Leeds & Barnard, Engrs., Consolidated, Los Angeles, Calif.

The elastic behavior of a laminated arch is dependent upon the load. The greater the load the smaller is its flexibility. This is a distinguishing characteristic.

The flexibility of an arch slice is described through its ellipse of elasticity or, in other words, through the moments of inertia of its elastic weights about the principal axes of gravity. The fundamental problem in the design of a laminated arch, therefore, is the determination of the effective moment of inertia of its elastic weights which is limited by the elastic behavior of the most flexible, up-stream lamina as a maximum and the elastic behavior of the solid arch with a thickness equal to n laminæ as a minimum.

Assuming for the moment a symmetrical layout and numbering the laminæ consecutively, beginning at the up-stream face, the elastic center, elastic weights, and moments of inertia of Arches 1, 1 + 2, 1 + 2 + 3, etc., are determined.

Their centers of gravity are readily found, the principal axes are located, and the moments of inertia about these axes are determined. The effective moment of inertia of the combined laminæ is $\frac{1}{n}$ times the sum of the moments of inertia of the individual groups (1, 1 + 2, 1 + 2 + 3, etc.), because each of the n combinations contributes $\frac{1}{n}$ parts of its flexibility to the elastic behavior of the entire section.

In Fig. 17 this hypothesis is illustrated for a symmetrical section with three laminæ of equal thickness. It shows clearly the diminishing of the ellipse of elasticity; that is, the decrease of flexibility as the number of laminæ engaged to carry the load increases. (Arches 1, 1 + 2, and 1 + 2 + 3 are designated *A*, *B*, and *C*, respectively, in Fig. 17.)

The elastic weights, W, of Arches *A*, *B*, and *C*, are:

$$W_A = \frac{24\left(R_u - \frac{t}{2}\right)\phi_0}{E\,t^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

$$W_B = \frac{3\,(R_u - t)\,\phi_0}{E\,t^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

and,

$$W_C = \frac{8\left(R_u - \frac{3\,t}{2}\right)\phi_0}{9\,E\,t^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

The distances, y_s''', of the elastic centers from the common center of curvature, C, are:

$$y_A = \frac{\left(R_u - \frac{t}{2}\right) \sin \phi_0}{\phi_0} \qquad (16)$$

$$y_B = \frac{(R_u - t) \sin \phi_0}{\phi_0} \qquad (17)$$

and,

$$y_C = \frac{\left(R_u - \frac{3\,t}{2}\right) \sin \phi_0}{\phi_0} \qquad (18)$$

The moment of inertia of the elastic weight, W (about the x-x axis), was found graphically as $E\,I\,W = 3.33$ (see Fig. 17).

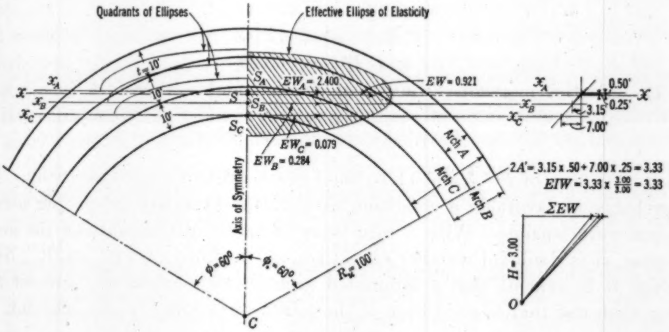

Quadrants of Ellipses
Effective Ellipse of Elasticity
t = 10'
10'
10'
x_A
x
x_B
x_C
S_A
S
S_B
S_C
$EW_A = 2.400$
$EW = 0.921$
$EW_C = 0.079$
$EW_B = 0.284$
Arch A
Arch C
Arch B
Axis of Symmetry
$\phi = 60°$
$\phi_0 = 60°$
$R_u = 100'$
C
0.50'
0.25'
3.15'
7.00'
$2A' = 3.15 \times .50 + 7.00 \times .25 = 3.33$
$EIW = 3.33 \times \frac{3.00}{3.00} = 3.33$
ΣEW
H = 3.00
O

FIG. 17.

The moments of inertia of Arches A, B, and C (about the x-x axis), are (very approximately),[50]

$$I'_A = \frac{24\left(R_u - \frac{t}{2}\right)^3}{E\,t^3} K' \qquad (19)$$

$$I'_B = \frac{3\,(R_u - t)^3}{E\,t^3} K' \qquad (20)$$

$$I'_C = \frac{8\left(R_u - \frac{3\,t}{2}\right)^3}{E\,t^3} K \qquad (21)$$

in which,

$$K' = \frac{\phi_0}{2} + \frac{\sin 2\,\phi_0}{4} - \frac{\sin^2 \phi_0}{\phi_0}$$

---

[50] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), top of p. 1675.

About the *y-y* axis, the moments of inertia are:

$$I''_A = \frac{24\left(R_u - \frac{t}{2}\right)^3}{E\,t^3}\,K'' \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

$$I''_B = \frac{3\,(R_u - t)^3}{E\,t^3}\,K'' \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

and,

$$I''_C = \frac{8\left(R_u - \frac{3\,t}{2}\right)}{9\,E\,t^3}\,K'' \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

in which,

$$K'' = \frac{\phi}{2} - \frac{\sin 2\,\phi_0}{4}$$

Introducing numerical values as shown in Fig. 17 there results:

$$\begin{aligned} E\,I'''_A &= 493.00 \\ E\,I'''_B &= 52.35 \\ E\,I'''_C &= 13.07 \\ \Sigma\,E\,I''' &= 558.42 \end{aligned}$$

The effective moment of inertia of elastic weights, therefore, is:

$$I' = 3.33 + \frac{558.42}{3} = 189.47$$

which is 38.30% of I'''_A, and 14.45 times greater than I'''_C, respectively. This is probably as favorable a condition as could be expected, if spacing between laminæ were feasible. With a thin layer of asphaltum separating the arches, however, it is doubtful whether an equivalent flexibility would result. Nevertheless, it is evident that a laminated arch shows a materially greater flexibility than one that is solid even if the joints were 50%, or less, efficient.

F. W. Hanna,[51] **M. Am. Soc. C. E.** (by letter).—The paper involves interesting questions in arch dam design: As to the superiority of stress analysis for the thin arch ring; the layout problem for dams in irregular canyons; the use of forked abutments to decrease arch spans; the effect of overhang on the layout and stresses; and the substitution of a series of thin arches for a single thick one.

Designers of arch dams are familiar with the difficulties attending the analysis of thick arch rings near the bottom of high dams. When arch formulas are used, tension appears to be involved, both at the crown and at the abutments; while, as a matter of fact, it is probable that combined arch and beam action exist in a complicated form. Undoubtedly, on account of the shortness of the span and depth of the beam, shear provides the bulk of the required resistance. Even if the arch rings at the base were laminated, it is probable that shear along the canyon bottom and walls would still furnish the principal resistance.

The author proposes to use a cone layout for arch dams, but this involves an intricate manipulation of central angles, radius lengths, and overhang.

---

[51] Chf. Engr. and Gen. Mgr., East Bay Municipal Utility Dist., Oakland, Calif.

Up-stream conic surfaces of arch dams must consist of sections of the surfaces of parent cones. Such sections are parallelogrammatic, trapezoidal, triangular, parabolic, or hyperbolic in nature and are roughly the same as the possible canyon sections suitable for arch dam sites.

In the case of a parallelogrammatic section, the conical surface will be cylindrical as the cylinder is a cone with its vertex at infinity. Both the central angle and radii from top to bottom of the dam can be kept constant, and difficulty with overhang can be obviated. In the case of a triangular section, the vertex of the cone will be at foundation level, and the central angle can be kept constant from top to bottom of the dam, the radii will decrease uniformly from the top to the bottom of the dam, and difficulty will be experienced with overhang. In the case of a trapezoidal section, the vertex of the cone will be below foundation level, the central angle can be kept constant from top to bottom of the dam, the radii will vary uniformly from a maximum at the top to a minimum at the bottom of the dam, and difficulty will be experienced with overhang. In the parabolic or hyperbolic section, the vertex of the cone will be below foundation level, both the central angle and the radii will decrease from the top to the bottom of the dam, and difficulty will be experienced with overhang. Consequently, the up-stream surface of the dam must generally either be warped or subject to overhang.

It is also desirable to have all the crown points and arch ring centers in the same plane and, if possible, to have the plane vertical. If the cross-section of the dam site is an isosceles triangle, an isosceles trapezoid, an equi-angular parallelogram, or any symmetrical curve with respect to a vertical axis, all the crown points in the neutral axes of the arch rings and the centers of curvatures or of symmetry of the neutral axis will lie in a vertical plane. If the triangle or trapezoid is oblique, the parallelogram non-equi-angular, or the curve symmetrical in horizontal planes with respect to an oblique axis, then the plane containing the crown points of the neutral axes and their centers of gravity or of symmetry will be in an oblique plane. In either case the horizontal arch rings will be symmetrical and symmetrically loaded.

If, however, on account of irregularities in the sides of the canyon walls, the cross-section does not conform to the rectilinear boundary of the triangle, trapezoid, or parallelogram, or to the boundary of a curve symmetrical in horizontal planes with respect to an oblique plane, the various arch rings cannot be made symmetrical with reference to a single vertical or inclined plane. If the distances of the crown points at the neutral axes or their centers of curvature from a vertical or an inclined plane are small, only slight errors will be involved in considering them symmetrical and symmetrically loaded.

As stated by Mr. Noetzli, the use of forked abutments in the design of arch dams will promote general symmetry of the dam and will decrease the span and the radii in the upper section.

The excessive weathering of the canyon walls near the top of most dam sites so increases the width as to make the design of the upper part of a high arch dam either uneconomical or infeasible without the construction of abutments to take the place of the missing canyon walls. In order to meet the walls in wide canyons at suitable angles the arch has to be too flat for economy;

and, if tangential abutments of full height are used, they are sometimes so nearly parallel to the canyon walls as to make them miss the walls entirely or to make their dimensions uneconomical. To overcome this difficulty the designers of arch dams have used the principle of forked abutments, the up-stream fork serving in part as a gravity dam of short length to close the reservoir and also sometimes in part as a buttress to resist arch thrust, and the down-stream fork serving only as a buttress to take the remaining thrust of the arch. Usually the buttresses are designed with outward sloping tops so that the weights when combined with the arch thrusts taken by them at all points from the springing lines outward meet the adjacent horizontal planes or foundations at angles less than the friction angle. Whether the inclined arch action should be considered in the design of these buttresses is somewhat questionable, as the amount carried by them is not easily determined and their extra cost due to embodying this additional factor of safety is usually not too great.

Forked abutments somewhat similar to those suggested by the author have been used with success in the Santeetlah Dam and the Calderwood Dam built by the Aluminum Company of America. The writer used this principle also in the preliminary design of an arch dam for the Pardee Dam site for the East Bay Municipal Utility District, Oakland, Calif.

In the Santeetlah Dam the sloping buttresses of the forked abutments were made radial to the arch section at the springing lines; they were heavy enough to take the radial thrust and to throw the remaining thrust of the arch into direct line with the other forks of the abutments; these, in turn, were designed to take this thrust and, as gravity sections, to close the reservoirs at the ends of the arch. In the Calderwood Dam it was found more convenient and economical to use three forks for the abutments. The first forks consist of gravity sections to close the reservoir, running on a radial line from the springing lines at the left abutment and nearly so at the right abutment; the second forks consist of sloping spray walls running along radial lines from the springing lines, sloping downward at a 0.5 to 1 slope, and designed to take the radial thrust and to act as spray walls; the third forks consist of buttresses extending from the ends of the arch in the line of the remaining thrust and sloping downward away from the arch at a 2 to 1 slope. They are designed to take the normal thrusts at the ends of the arch. In the preliminary design of the arch dam for the Pardee Dam site, the forked abutments were computed so that the gravity sections closed the reservoir by the shortest routes to the canyon walls, and the buttresses were designed to slope downward away from the ends of the arch at a 2.5 to 1 slope and to take the entire thrust of the arch.

The effect of overhang down stream at the crown will be beneficial in the control of the central angles and radii, and in carrying the load to the abutments by inclined arch action. However, it complicates the arch ring analyses. An equally important help can be obtained in the adjustment of the radii and central angles through overhang in an up-stream direction adjacent to the abutments. Abutment overhang, however, inclines the arch rings upward and carries the stresses upward at the abutments. This overhang usually occurs near the base of the dam, however, where the weight of the superstructure and the shortness of the arches counterbalance the bad effect. Advantages are obtained, however, by a combination of an overhang down stream at the crown near the top and an overhang up stream at the abutments near the base of the dam; but the overhang should never be enough to endanger the gravitational stability of the structure.

The writer is not fully convinced that proper transmission of stresses can be made across the laminations in the laminated type of dam. In the first place, imperfections in construction may be such as to cause the load from any lamination to be improperly transmitted to, and distributed on, the next succeeding lamination down stream. The arch rings under water load deflect down stream at the crown and up stream near the abutments. The deflection of the primary arch under water load down stream at the crown will transmit a certain load to the next succeeding arch at the crown, but not necessarily at the abutments. Consequently, the loads on the secondary arches will have different distributions and the deflections of the secondary arches will not conform accurately to those of the primary arch and to each other. These inequalities in loads and deflections will upset the stress calculations and make them uncertain both in the primary and in the secondary arches.

As a matter of fact, the lack of conformity in load distribution and deflections seems to be a serious defect in the laminated arch type of dam. Further, there is the possibility that cracks will form in the thin laminations of the laminated dam, as shown by the experience in the Stevenson Creek Experimental Dam. Moreover, if there is open space between the laminations due to shrinkage in the concrete and to settling under gravity of the asphaltic filling, undue strain may result in the primary arch. Over-deflection of the primary arch near the top of the dam may cause uplift and cracking through strut action in the secondary arches. On the other hand, if insufficient lubrication exists between the laminations, they will not act as separate arches; as a matter of fact, they cannot act fully independently unless there is a frictionless surface between them.

A. A. EREMIN,[52] ASSOC. M. AM. SOC. C. E. (by letter).—The author has indicated a number of limitations of the present methods used in designing thick arch dams due to the "uncertain assumptions" on which computations of the unit stresses are based. In order to simplify doubtful complicated mathematical manipulations in the design of arch dams the author introduces a new type of laminated arch dam composed of thin arches which are separated from each other by sliding joints or films of water. Since the unit stresses in sections of thin arches are more uniform than in thick arches the error in determining stresses in the laminated arch dams by the methods based on the cylindrical formula will be less.

The idea of subdividing compressed thick arch dams is not new; it was proposed in 1894 by the French engineer, Boulé; in 1912 by the Italian engineer, Rutenberg; and, recently, it was applied to actual practice in the French dam on the Dordogne River as stated by Mr. Noetzli.

---

[52] Asst. Designing Engr., Bridge Dept., State Highway Comm., Sacramento, Calif.

The horizontal deflections of thick arch dams are generally small and their effect on the primary stresses in an arch is commonly neglected. However, in thin arches, deflections are greater and their influence on the stability of the arch must be considered.

For a circular arch with hinged ends acted upon by a uniform radial pressure Professor S. Timoshenko developed the following expression for critical pressure or the maximum pressure beyond which a structure might buckle:[53]

$$q_{cr} = \frac{E\,I}{R^3}\left(\frac{4\,\pi^2}{4\,\phi_1^{\,2}} - 1\right) \qquad (25)$$

It is evident from this formula that the buckling effect in thin arches is inversely proportional to the ratio, $\frac{R^3}{I}$, or $\frac{R}{t}$. Furthermore, in concrete arch dams with central angles of less than 90°, the buckling of the structure need not be feared.

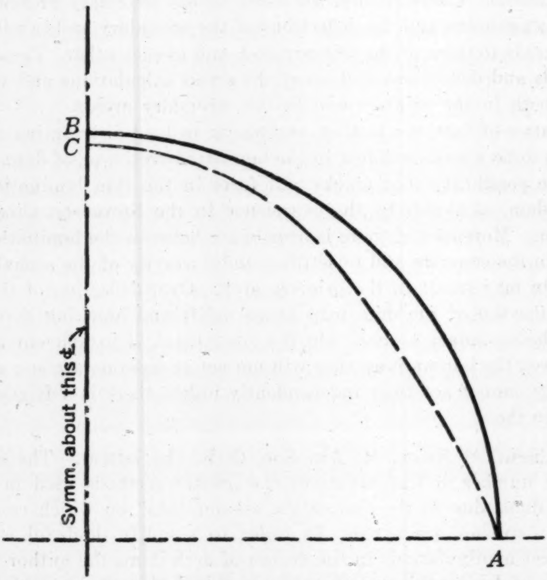


FIG. 18.—RELATIVE DEFLECTION OF AN ARCH DAM SUBJECTED TO A UNIFORM RADIAL PRESSURE.

The author has stated that, neglecting the effect of cantilever action, "the deflections of corresponding points along radial lines will be the same for all arch laminæ." Within practical limits, this statement is correct when it refers to the upper parts of thin arches that are fixed at the bottom. However, near the bottom, deflections vary along the axis of the arch, thus inducing a variation in the distribution of the external pressures carried by arches unless a film of water is provided between laminæ.

---

[53] "Applied Elasticity," by S. Timoshenko and J. M. Lessels, First Edition, 1925, p. 247.

Professor C. Guidi made an experimental test to determine the deflection of a circular concrete arch, 16 ft. 4¾ in. high, 27 ft. 10½ in. long, and 6¼ in. thick, with a central angle equal to 180 degrees.[54] This arch was fixed at the bottom and both ends, and was subjected to a uniform radial pressure. In Fig. 18 the curve, *AC*, represents the deflection of this arch at Elevation 5 ft. 9 in. Its original unloaded position is represented by the curve, *AB*.

The effect of the variation in deflection of thin arches in the laminated arch dam on primary stresses is probably small. However, it might be rapidly increased with an increase in the number of laminæ used in the dam.

The advantages of using forked abutments have been well indicated by the author. However, when there are certain topographical conditions at a dam site, such that a high, gravity type of wing-wall would be required, the French buttresses with arched flanks, as shown in Fig. 9, would be more suitable.

The arched flanks are more economical than the high, gravity type of wing-walls and the stresses in the buttresses are symmetrically distributed about the center line of the arch. Therefore, the buttresses may be of plain concrete. Steel reinforcement in the main structural members of dams is frequently objectionable because of possible corrosion.

By inclining the center lines of buttresses toward the center line of the main arch in the French type of laminated dam efficient security against sliding is obtained.

The Engineering Profession should appreciate highly the author's development of this theory of designing arch dams and his bold application of the scientific principles to actual practice.

Fred A. Noëtzli, M. Am. Soc. C. E. (by letter).—There are several more or less independent features of arch dam form and layout described in this paper, namely, the so-called "forked abutments", a pronounced sloping of the crown of the arch, and the subdivision of the arch, where indicated, into two or more arch laminæ. Furthermore, a simple and convenient method is given whereby the arch stresses at the crown and the abutments, due to uniform water pressure or change of temperature, can be computed from the axial stress (cylinder formula) by mere multiplication with a coefficient to be taken from diagrams given in the paper. Similar diagrams are given for coefficients computed by the theory of "thick" arches.

The three features described in the paper may be used either singly, entirely, or in combinations of two. An arch slightly overhanging in its upper central portion will be found more economical in most cases and will have a better distribution of stresses than an arch dam of the ordinary type.

Forked abutments on one or both sides of the arch will be advantageous at most dam sites. As mentioned by Mr. Hanna, they have been used with success in the Santeetlah and Calderwood (arch) Dams; and the Stewart Mountain (arch) Dam on the Salt River, in Arizona, has forked abutments.

The writer agrees with Mr. Jorgensen that in fairly thin dams the arch should not be laminated. The greatest benefit from laminating is obtained in a relatively thick structure. To the extent that they are effective, the laminations tend to give a better stress distribution in the vertical cantilever elements as well as in the horizontal arches. They also improve the distribution of the foundation pressures both under the vertical cantilevers and, laterally, at the arch abutments.

---

[54] *Revue Générale de l'Electricité*, October 29, 1927.

Mr. Jorgensen and Mr. Hanna appear to place considerable reliance on shear resistance for carrying the load in the lower portions of an arch dam. Consideration should be given in this connection to the fact that, at the support of an ordinary cantilever or beam of plain concrete, the shear stresses may be relatively small while the compression or tension stresses from bending may be large enough to produce failure. Small shear values alone, regardless of their magnitude, can not be relied on to insure the stability of a structure, such as a beam, a cantilever, or an arch, unless the tension and compression stresses are also within proper limits.

It is gratifying that, by a different process from the one used by the writer, Professor Cain arrived at the same equation from which the coefficients of Figs. 1 and 2 were computed. A further check is furnished by comparison with Figs. 3 and 4 for "thick" arches. The writer is in full accord with Professor Cain regarding the desirability of vertical radial contraction joints in arch dams. Such vertical joints were purposely omitted in the Stevenson Creek Experimental Arch Dam for the main reason that it was desired to test horizontal arch elements which would be continuous from abutment to abutment so as to permit the measurement of continuous strain curves between abutments without the presence of disturbing irregularities at such joints. Although in the course of the experiment several cracks developed in the dam—particularly a vertical crack in the center extending about 10 ft. from the crest downward and another crack in the same vertical plane extending from the base upward for a distance of about 13 ft.—fortunately none of these crossed the horizontal strain-gauge line at Elevation 30. The continuity of the strain-gauge line at this elevation, therefore, remained unimpaired. The measurement of strains along this line furnished some of the most interesting results of the Test Dam investigation.[55]

There is apparently a misunderstanding regarding the meaning of the term, "rib-shortening" stresses, as used in the derivation of Equations (1) to (5) for the total stresses resulting from the shortening of the arch rib, that is, for direct and bending stresses combined. Professor Cain's Equation (12) gives the direct stresses only.

Mr. Jakobsen and Dr. Vogt present interesting computations relative to the effect of friction in the joints between the arch laminæ. A certain amount of friction will be unavoidable. It is evident, however, that the worst that could happen is that the friction would prevent all movement. The structure then would act simply as a solid arch. However, any movement in a joint would tend to diminish the arch stresses. The extent of this re-adjustment of stresses is in proportion to the tangential movement in the joints. In this connection it is of importance to note that deformations of a laminated arch due to shrinkage and changes of temperature involve very small radial stresses and, consequently, develop very little, if any, friction in the joints. For such deformations, therefore, an adjustment of the stresses could take place approximately as indicated by theory. If water pressure, shrinkage of concrete, and change of temperature of the arch occur in combination with each other, as must usually be assumed for the critical stress conditions, the radial pressure upon the joints remains practically the same as for water pressure alone, while the shear, under certain conditions, may be doubled or tripled, and, accordingly, also the tendency of sliding in the joint.

---

[55] Report on Arch Dam Investigation, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 164.

Mr. Jakobsen recommends the grouting of contraction joints to counteract the shrinkage of the concrete and to improve the stress conditions in arch dams. He states that "this method has been used in connection with a number of dams and apparently with considerable success." The writer has had occasion to get first-hand information on, and, in a few cases, to be directly connected with, such grouting operations on several dams. This experience leads him to believe that the result of grouting of contraction joints in arch dams is very uncertain, to say the least. There is the danger of some portions of the area of a joint being grouted and others not, which results in concentrations of pressures over the grouted areas when the dam is loaded. In the Pacoima (arch) Dam the quantity of cement grout forced through the grout pipes into the contraction joints, under a pressure of 100 lb. per sq. in., was less than that which would have been required for filling the pipes. This indicates that some of the pipes were at least partly clogged by "fines" or other materials during the construction of the dam. Under such conditions it is very doubtful just how much grout was actually forced into the joints themselves and how it was distributed over the section.

The writer agrees with Mr. Werner that it is important to secure monolithic action in the forked abutments. Steel reinforcement placed cross-wise between the buttress and the gravity wing, as indicated in Fig. 7, will promote such monolithic action. Some extra concrete placed in the trough between the buttress and the gravity wing will further improve it. In order to avoid the disadvantages due to shrinkage of the concrete, as pointed out by Mr. Werner, it will be advisable to keep the buttresses as low and as short as possible. As far as the writer knows no difficulty has ever been experienced in arch dams abutting against gravity tangents.

Dr. Vogt refers to theoretical and experimental deductions which indicate that the thrust of the uppermost arches is inclined at the abutments.

Since the crown of the dams shown in the paper is sloping, the actual load-carrying arch elements will be inclined downwardly at the abutments considerably more than in the dams investigated by Dr. Vogt. This is one of the important features of the design as it permits with safety the use of higher forked abutments and consequent shorter radii and spans in the central arch. The most economical layout is best determined by trial. For the uppermost arches it is of advantage to use a relatively long radius and a relatively small central angle so as to direct the arch thrust "into" the hillside rather than parallel to it, with consequential beneficial results as regards the stability of the forked abutments. For practical reasons these arches are thicker than would be theoretically necessary, and, therefore, the central angles need not be very large to give stresses within permissible limits.

Regarding materials for decreasing friction in the joints between the arch laminæ the writer is emphatically against the use of any substance which—either because of the composition or because of the thickness of the layer—might flow to a considerable extent. He believes that a thin coat of asphaltic paint or some of the other materials mentioned in the paper will be sufficient to enable movement in the joint leading to at least a partial re-adjustment of the arch stresses with consequent beneficial results. The writer is grateful to Dr. Vogt for having emphasized the probability of the up-stream laminæ being stressed relatively more and taking a larger portion of the load than the down-stream laminæ. Dr. Vogt gives as reasons the expansion of the up-stream laminæ due to water-soaking, and a smaller value of a "sustained modulus of elasticity" for continuous load on the dry lower laminæ. In a "thick" arch of a dam the highest stresses occur at the abutment intrados. Along the entire extrados the stresses are relatively small. As an example, consider the arch assumed by Mr. Jakobsen for which $\frac{t}{r} = 0.50$ and $2\phi = 120$ degrees. Assume this arch to be stressed by an axial compression of 250 lb. per sq. in. (cylinder formula). From Figs. 3 and 4, the coefficient, K, for the given values of $\frac{t}{r}$ and 2ϕ, was obtained for use in computing the arch stresses. The cylinder stress, multiplied with the values, K, gives the stresses in the arch at the crown and abutment directly, as shown in Table 3.

TABLE 3.—COMPUTATIONS FOR DETERMINING ARCH STRESS.

Point of arch.	Axial stress, in pounds per square inch.	K.	Arch stress, in pounds per square inch.
Extrados: Crown	250	1.08	270 (compression)
Extrados: Abutment	250	−0.38	−95 (tension)
Intrados: Crown	250	0.12	30 (compression)
Intrados: Abutment	250	2.52	630 (compression)

Evidently, in the solid arch, the concrete near the extrados is used very inefficiently, the stress being 270 lb. per sq. in. compression at the crown and 95 lb. per sq. in. tension at the abutment, while the greatest stress in the arch is 630 lb. per sq. in. compression at the abutment intrados. Consequently, if subdividing such an arch into two or three laminæ tends to put more load and a correspondingly greater stress on the up-stream arch for the reasons given by Dr. Vogt, the results can be only beneficial. If, in this example, a drop of temperature and shrinkage of concrete were considered, the comparison would be still more strikingly in favor of the laminated type.

If a reliable method of grouting of contraction joints of arch dams were available the writer would recommend that the joints in the up-stream arch be grouted under a higher pressure than those of the down-stream arches. As a result, the up-stream arch would take more stress and, therefore, carry more load than the corresponding parts of the solid arch. Naturally, certain other parts of the arch would be relieved by the same amount of load. Some overstressing of the up-stream arch within reason would be beneficial rather than detrimental to the safety of the structure as a whole, inasmuch as it would lead to a better distribution of the foundation pressures because the thrust of the up-stream arch would engage the up-stream portions of the side abutments which in the solid arch are stressed very little and may even be theoretically under tension. It also should be borne in mind that there is not the remotest possibility of independent failure of the up-stream arch because it could not break through without deflecting and thereby engaging the down-stream arches. This, of course, would relieve the up-stream arch immediately of the assumed excessive load.

Mr. Bauman shows by the ellipse of elasticity the greater degree of flexibility of a laminated arch as compared to that of a solid arch, even if the joints should be only 50% effective. Greater flexibility results in a decrease of the bending stresses due to rib-shortening, temperature drop, and shrinkage, and a corresponding greater efficiency of the arch. Mr. Bauman suggests the possibility of slips between the laminæ. For the arch shown in Fig. 8 a maximum movement of 0.06 in. was computed for a change from no load to full load. It needs a rather vivid imagination to speak of "dam quakes" for such small movements.

Mr. Hanna gives an interesting contribution to the discussion in the description of several arch dams in which some of the characteristics of the type of dam described by the writer have been used to advantage. He also calls attention to the feasibility of an overhang of the arch in an up-stream direction adjacent to the abutments. Mr. Hanna expresses doubt regarding the proper transmission of loads across the lamination joints. In this connection, it is of importance to consider that the second lamina will be cast tightly against the first which previously may have been built one or two lifts higher. Before pouring the second lamina the adjoining face of the first will be painted with asphalt so as to prevent bonding and decrease friction. Nevertheless, the two laminæ will be in direct contact with each other at all points. The effect of shrinkage of the concrete in a transverse direction may tend slightly to open the joint, while on the other hand the overhang of the dam will have the tendency to keep it closed.

Assume the water to rise slowly in the reservoir. It will exert a uniform pressure against the up-stream lamina. In the case of two laminæ in contact with each other both will deflect equally. If the deflection lines of two arches are of practically identical shape (as are the up-stream and the down-stream laminæ of an arch dam), the loads causing these identical deflections must also be practically identical. The water load on the arch, therefore, is divided equally between the two laminæ. If, as assumed by Mr. Hanna, the down-stream arch deflected over a certain distance more than the up-stream arch, the joint would open naturally. As no load can be transmitted across an open joint the up-stream arch would have to carry the total water load, while the down-stream arch would be without load over this distance. Two or three determinations of arch deflections under these assumptions will quickly convince even the most skeptical that there is no possibility of such conditions arising. Therefore, if the joint does not open, the deflections of the two arches —and also the loads and stresses—must be identical, at least within the limits of similarity between the up-stream and the down-stream arch. The writer, therefore, is unable to agree with Mr. Hanna that "the loads on the secondary arches will have different distributions and the deflections of the secondary arches will not conform accurately to those of the primary arch and to each other."

Mr. Eremin cites Professor Timoshenko's equation for the maximum pressure beyond which an arch might buckle. Thin dams in which $\frac{t}{r}$ is not more than, say, 0.15, require no laminating and in thick dams there will seldom be need of more than one or two joints, so that the question of buckling will scarcely ever arise in this type of structure.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

TRANSACTIONS

Paper No. 1775

THE SHANNON POWER DEVELOPMENT IN THE IRISH FREE STATE[1]

BY A. R. C. MARKL,[2] JUN. AM. SOC. C. E.[3]

WITH DISCUSSION BY MESSRS. L. J. BEVAN, AND A. R. C. MARKL.

SYNOPSIS

This paper aims to present a broad outline of the civil engineering part of the construction of the Shannon Power Development in the Irish Free State. The work was done by German contractors and the construction methods used are representative of the best European practice.

Inasmuch as the manner of dealing with the concrete and rock work did not differ appreciably from American practice under similar circumstances, this part of the description will be restricted to a brief recital of the features peculiar to this particular construction.

The equipment and methods used in building a large waterway, such as the head-race of the power plant, however, present features which are not only fundamentally different from American practice, but which are also without precedent in Europe. With the enormous task of building levees on the Mississippi River looming before American engineers, it is thought that a detailed account of the methods used in excavation and in the building of embankments should prove of particular interest.

GENERAL DESCRIPTION OF THE DEVELOPMENT

Southern Ireland with its 25 000 sq. miles and 3 000 000 inhabitants has very little industry. It uses about 45 000 000 kw-hr. per year; that is, only 15 kw-hr. per person per year as against 650 kw-hr. in the United States. This power is supplied by small local stations at prices ranging as high as 20 cents per kw-hr. When the Irish Free State achieved its independence, the Govern-

---

[1] Published in April, 1930, *Proceedings*.

[2] Chicago, Ill.

[3] Now Assoc. M. Am. Soc. C E.

ment recognized as one of its foremost tasks the development of the country's latent power resources, with a view to infusing new vigor into its economic life. As early as 1925 the Shannon Electricity Act was passed by the Oireachtas, authorizing the construction of a water power plant on the River Shannon. It is Government-owned, and will supply a minimum of 150 000 000 kw-hr. in 1930, and a maximum of 500 000 000 kw-hr. in a later, more complete, development. The cost of the first part of the work, exclusive of the transmissions and transformer stations, was $16 500 000. Basing estimates on amortization in 40 years and 5.5% interest, and including operating expenses and over-head, the cost per unit at the power house will be 0.80 cent for the first part of the development, and 0.45 cent for the full development. The corresponding cost at the 38-kv. bus-bars at Dublin, Cork, and Limerick, which cities are interconnected by 110-kv. loops, is estimated at a little more than 1 cent per kw-hr.

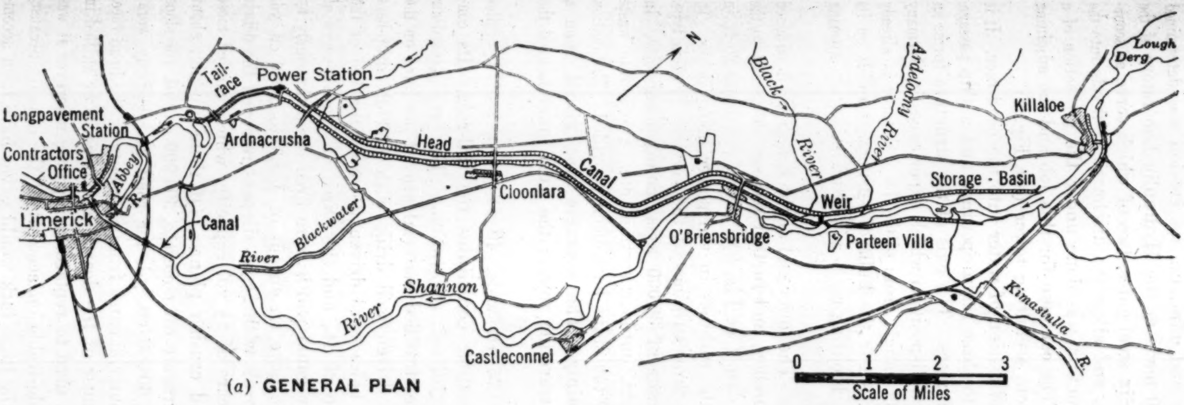
The conditions for the construction of a large water power plant near Limerick may be considered as favorable. The River Shannon is about 200 miles long, and on its course through the center of the island drains about one-sixth its total area, or 4 000 sq. miles, with a mean annual rainfall of 37 in. The average flow in its lower reaches is 8 500 sec-ft., the highest floods carry 32 500 sec-ft., and the lowest run-off observed was 900 sec-ft.; the ratio of the two latter values is 36 : 1, which compares very favorably with most European rivers. The upper course of the Shannon has very little slope, since the river flows through three extensive lakes, Lough Allen, Lough Ree, and Lough Derg, with a total surface area of more than 100 sq. miles, which serve as natural storage basins. The greatest drop is concentrated between the last-named lake and the tidal estuary of the river, and here it is possible to utilize a head of 100 ft. in a stretch of 17 miles.

PRINCIPAL CONSTRUCTION ITEMS

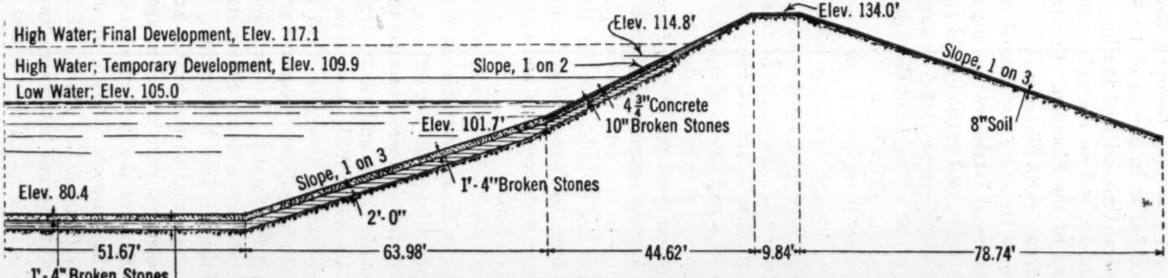
The actual work which had to be undertaken may be subdivided into the following main sections: (1) Raising the banks of the lower lake (Lough Derg) to increase its storage capacity; (2) construction of a diversion dam and canal head-gates; (3) head-race construction; (4) construction of the water-gate, the power station, and ancillary, or secondary, works; and (5) tail-race excavation.

Storage Capacity Increase.—Despite their large area, the existing natural lakes were insufficient to provide the storage space required for balancing the water supply between periods of flood and drought. Low embankments had to be thrown up along the sides of the lower lake, and parts of the Upper Shannon had to be regulated. Incidentally, these measures will benefit agriculture by protecting large areas from inundation, and shipping by providing a deeper and straighter navigable channel. This work was done mainly by means of small bucket excavators.

Diversion Dam and Head-Gates.—Six miles below Lough Derg (Fig. 1(*a*)), a short distance down stream from a geological fault crossing the Shannon at right angles, a dam which raises the water 25 ft., was built on the crest of the hard red sandstone stratum sloping up against the river at this point.


Lough Derg
Killaloe
N
Black River
Ardeloony River
Storage - Basin
Weir
Parteen Villa
Kimastulla R.
O'Briensbridge
Canal
Head
Cloonlara
Tail-race
Power Station
Ardnacrusha
Longpavement Station
Contractors Office
Abbey R.
Limerick
Canal
River Blackwater
River Shannon
Castleconnel
0 1 2 3
Scale of Miles

(a) **GENERAL PLAN**


High Water; Final Development, Elev. 117.1
High Water; Temporary Development, Elev. 109.9
Low Water; Elev. 105.0
Elev. 114.8'
Slope, 1 on 2
Elev. 134.0'
Slope, 1 on 3
8"Soil
4¾"Concrete
10"Broken Stones
Elev. 101.7'
Slope, 1 on 3
1'-4"Broken Stones
Elev. 80.4
2'-0"
51.67'
63.98'
44.62'
9.84'
78.74'
1'-4"Broken Stones
2'-0"Puddle Lining

(b) **TYPICAL HALF-SECTION OF THE HEAD-RACE OF THE POWER HOUSE**

FIG. 1.—PLAN AND SECTION OF THE SHANNON POWER DEVELOPMENT.

This dam is of the ogee overflow type, and has an over-all width of 380 ft.; the form producing favorable flow and minimum toe erosion was determined by model tests in a scale of 1 : 50 made at the Hydraulic Laboratory of the Technical University of Berlin. Six steel gates, two of which serve as scouring sluices, rest on the weir crest, and allow the discharge of floods into the old bed of the Shannon. Construction was done under the protection of a steel-frame earth-fill coffer-dam. The concrete for this job and the adjoining head-gate structure was poured from a chuting tower (see Fig. 2).

The canal intake governs the admission of water into the head-race. It is 320 ft. wide, and is subdivided into three main openings and a ship passage, all of which can be closed by steel gates. The axis of the structure forms an angle of 45° with that of the dam, a disposition which experiences in Germany have proved suitable for protecting canals from silt. This danger is already minimized inasmuch as the river sheds the bulk of suspended solids on its passage through the storage lakes. Floating matter is restrained from entering the head-race by a panel wall.

Water-Gate and Power Station.—The lower end of the head-race is widened so as to form a trumpet-shaped forebay, and further passage is barred to the water by a medium-sized concrete dam (see Fig. 3). Short penstocks 20 ft. in diameter lead from this dam to the turbines in the power house below. In the present development there are three turbines of the vertical Francis type, each directly coupled to a generator of 30 000 kv-a. nominal capacity. In addition to the actual power plant, outdoor station, etc., a concrete waste channel and a double lock for 150-ton ships have been provided. Excavation was done by power shovels mainly. The concrete was placed from a cableway with one fixed and one traveling tower; the fixed tower housed the mixing plant.

Tail-Race.—A channel, about 75 ft. wide, 40 ft. deep, and 1¼ miles long, cut into carboniferous limestone constitutes the tail-race. Its construction involved 800 000 cu. yd. of excavation. The compressed-air method was abandoned because the stratified material proved too hard on the pneumatic drills, and electrically driven well drills with 1 500-lb. drop-bars were adopted. With these, 8-in. holes were driven down to the bed of the future canal at intervals of 15 to 20 ft., and the three or four charges of ammoniated saltpeter required per canal width were fired simultaneously by electric detonators. This schedule gave an output of 4 000 to 5 000 cu. yd. of limestone per blast. Steam shovels with 2½-yd. dippers removed the débris and loaded it into bottom-dump cars of 8½ yd. capacity, which carried most of the material to the washer and crusher plants. Of their product, about 400 000 cu. yd. were needed as aggregate for concrete, 500 000 cu. yd. for slope protection for the head-race, and 60 000 cu. yd. for road and railway work, and other purposes. Part of the material came from rock cuts in the head-race.

The Construction Plant.—Because of the dearth of native labor skilled in construction work, machinery was used to replace man-power wherever it was possible to do so. This was also desirable on account of the climate because, with 270 rainy days in the year, the huge task would never have been accomplished in the scheduled construction period of 3½ years. To this end, 30 000

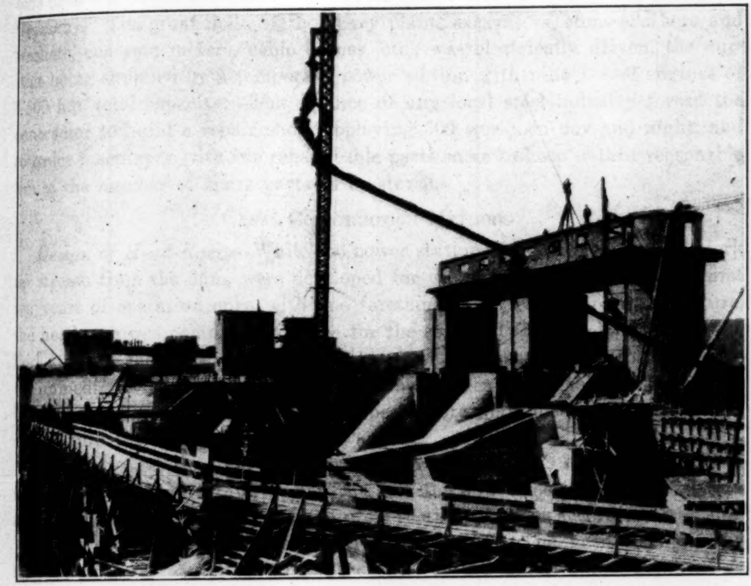


FIG. 2.—POURING CONCRETE FOR DIVERSION DAM. HEAD-GATES TO BE SEEN IN REAR.

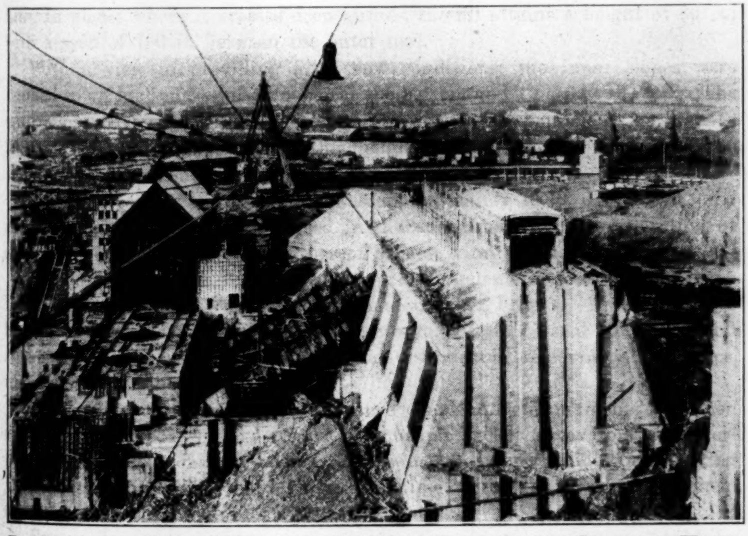


FIG. 3.—SHANNON POWER DEVELOPMENT: TAIL-RACE, POWER STATION, PENSTOCKS, WATER-GATE, FOREBAY OF HEAD-RACE. IN REAR, FIXED TOWER OF CABLEWAY.

tons of construction machinery, large and small, were shipped to Ireland from Germany. The great mass of the heavy plant, excavators, stone-crushers and washers, concrete mixers, cable cranes, etc., was electrically driven, the current being supplied by a temporary power station with nine Diesel engines of 4 500 h.p. total capacity. The absence of any local steel industry forced the contractor to build a repair shop employing 300 workmen day and night, and to order machinery with interchangeable parts so as to keep within reasonable limits the number of spare parts to be stored.

Canal Construction Methods

Design of Head-Race.—While the power station, the tail-race, and all work up stream from the dam, were developed for the anticipated needs of the first ten years of operation only, with due forethought for extension in the future, the head-race was completed at once for the so-called full development, or the final stage of the work. A policy of restricting construction of the head-race to immediate requirements would not have been justifiable economically, inasmuch as present gains would have been more than offset by future excess expenditure for enlarging the waterway.

The head-race, therefore, was built for a capacity of 19 500 sec-ft. Since it was also to serve as a navigation channel, the flow velocity was limited to a maximum of 5 ft. per sec., and the cross-section planned accordingly.

A typical half-section is shown in Fig. 1 (*b*). The head-race has the notable width of 320 ft. between the crowns of the banks, and a depth of as much as 37 ft. in the final development. Only a short length of the canal is entirely in "cut", most of it being constructed partly in cut and partly as fill. In places where it crosses deep gullies, the fill attains a height of 60 ft., with a span of 700 ft. between the outer toes.

With a view to providing good flow conditions, the inner slopes were banked to an inclination of 1 on 3 near the bottom and 1 on 2 at the top. The outer slope of 1 on 3 assured adequate stability for embankments as high as 25 ft. Where the fill exceeded this height, berms were provided, and the slopes were flattened to 1 on 4 and 1 on 5.

Since the total earth excavation was approximately 10 000 000 cu. yd., and similar quantities had to be placed in the enbankments, proper selection of equipment and good disposition thereof became vital problems.

Preparation of Site.—The greater part of Ireland is pasture land, each individual plot being fenced in by stone walls or hedges. These obstacles had to be removed first. Walls and tree stumps were blasted out, and small steam shovels on caterpillars did excellent service in removing stone heaps, underbrush, and hedges.

The area to be occupied by the future canal and embankments was then stripped of surface soil by means of multiple-bucket excavators on caterpillars with 35-ft. conveyor booms, and the earth dumped aside for re-soiling the embankment slopes. Usually, two machines worked in series, one of them stripping, and the other rehandling, the material.

Excavation.—After these preparations, the actual excavation could be undertaken. The question arose, as to whether to use power shovels or multiple-bucket excavators. Conditions were such that, at first, neither equipment had any apparent advantage over the other, and only after long and careful study of their respective merits was the decision given in favor of the latter.

To be sure, its ruggedness, mobility, and general usefulness make the power shovel a very valuable tool, especially where excavation is irregular in plan or section and high resistances are encountered; and, therefore, as a matter of course, shovels of various capacities were adopted for foundations and rock work, at the power house and dam sites, as well as in the tail-race.

In the head-race, on the other hand, a shallow trough had to be formed true to design dimensions, and, if possible, in one single operation. The multiple-bucket excavator was found to be the only machine capable of excavating the canal bed and slopes, including breaks in the slopes, without rehandling material or much finishing (see Fig. 4). It raises the excavated material to surface level, and thus eliminates the expensive track work and up-hill haul required in the case of a power shovel operating in the cut, an advantage which is further accentuated where clayey water-bearing soil is encountered. However, it must be conceded that the tenacious marley clay strewn with boulders, which formed the bulk of the excavation on the Shannon work, was no more favorable to the multiple-bucket excavators than to the power shovels. In fact, this material marks the limit of usefulness of this type of excavator; but, even so, with only 30 to 40% of its theoretical capacity, it proved to be the economical superior of the power shovel for this kind of work.

The following are the principal data for one of the excavators used: With a bucket capacity of ⅓ cu. yd., and 22 to 28 buckets discharged per min., the excavator capacity is 500 cu. yd. per hour theoretically. Under favorable soil conditions, 350 cu. yd. should have been an easy average, but actually no more than 200 cu. yd. were obtained on this job. The length of the bucket ladder is 90 ft., and the maximum depth of excavation is 58 ft. By changing the ladder rigging, the machine can also be made to scrape material down from elevations higher than the track, in which case a height of 46 ft. is the limit. The machine travels at a speed of 16 ft. per min. on the track, that is, in a direction transverse to the bucket chain, and is capable of climbing grades as steep as 2½ per cent. Its service weight is 220 tons.

Portable transformer wagons step the 17 300-volt current supplied by the auxiliary power station down to 3 000, which is the voltage of the main drive, and a transformer inside the excavator steps it down farther to 380 and 220 volts, the tensions at which the advancing drive, the winch drive, and the compressor motor work. The machine requires 150 h.p.

The following were the reasons for adopting electric drive in place of steam for this and other equipment. An electric multiple-bucket excavator uses 0.4 to 0.6 kw-hr. per cu. yd., while a steam-driven machine of the same capacity consumes 3 to 3½ lb. of coal. Considering the necessary haulage in the case of coal, and including the amortization of the auxiliary power plant and transmissions in the price of current, costs per unit excavated and loaded are about balanced; but electric drive has the advantages of higher over-load capacity, instantaneous readiness, and elimination of delays for firing up; it needs no stoker and, therefore, can be operated by one man; and it requires

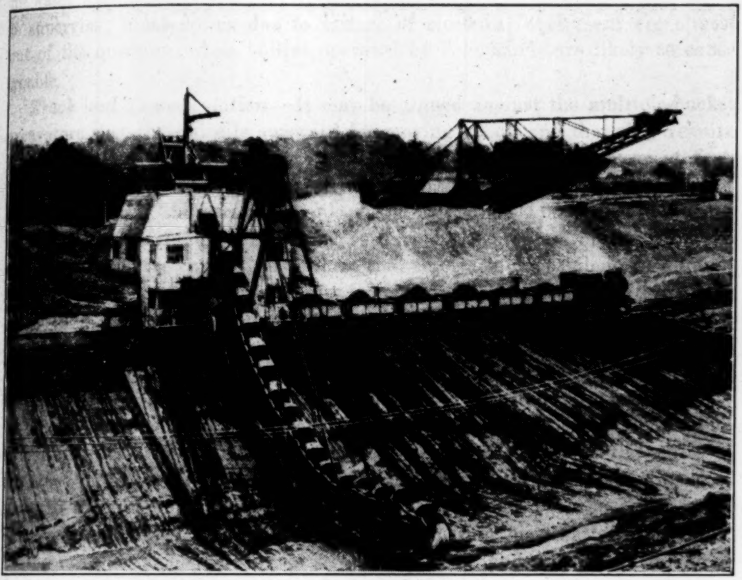


FIG. 4.—VIEW OF MULTIPLE-BUCKET EXCAVATOR FORMING HEAD-RACE, SHANNON POWER DEVELOPMENT. IN REAR, BANK-BUILDING MACHINE.

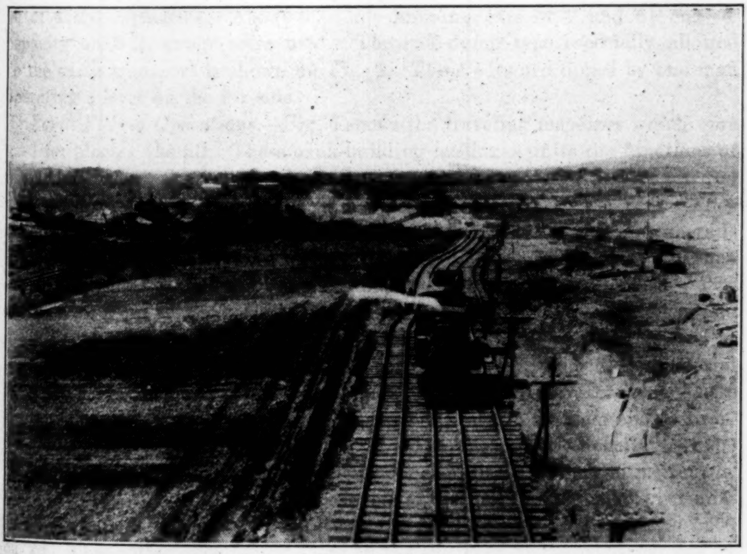


FIG. 5.—SHANNON POWER DEVELOPMENT: TRACK SHIFTER AT WORK.

no water stations and less lubricants. Furthermore, it is cleaner and easier to supervise; break-downs due to failure of electrical equipment are almost out of the question, while boilers operated by field hands are likely to cause trouble.

Track and Transportation.—It may be argued against the multiple-bucket excavators that their size is awkward for moving about, and that they require heavy track. Although true, this is not as serious as would appear at first. Transport track is necessary in any case, and this can be laid on top of the 20-ft. ties for the excavator rails. Not more than 1 000 ft. of excavator track are needed per unit, inasmuch as during the progress of the work the rear track units are carried forward, so that the track creeps along in a longitudinal direction. A greater difficulty is presented by the lateral shifting of the track in cases where the excavator chain is not long enough to complete the excavation from one position. Normally, a gang of forty men would be kept busy attending to this work, but with the mechanical track-shifter used no manual labor is required. Fig. 5 shows a track-shifter at work.

The machine consists of a steel truss, 50 ft. long, the ends of which rest on swiveling trucks which ride on the track to be slewed. A jaw at the center of the truss picks up the track at the rails and moves it in a lateral direction, so as to form a kink in the line. Nothing is disconnected, the entire track is moved, including 20-ft. ties and the transmission masts attached thereto. In the next position one kink is straightened out, and a new one produced farther ahead, and so on until the desired position is attained.

Corresponding to the large excavation units used throughout the work and the haulage distance of 1½ miles on the average, the rolling stock also had to be of large capacities. About 900 side-dumping cars of 7 and 8½ cu. yd. capacity on 3-ft. gauge were used. The side-dump type especially allotted to the earth transport is shown on Fig. 4. These cars are tipped by one man operating a lever on the far side.

Earth-Filling Operations.—Fig. 6 shows the traveling machines which were used for placing the fill. These bank-building machines unite the functions of a multiple-bucket excavator and a belt conveyer. They weigh 240 tons, travel at a speed of 27 ft. per min., and climb grades as steep as 3⅓ per cent. They run on four rails laid on ties 20 ft. long, which also accommodate the rails for the earth transport trains. The latter dump their material into a trench about 8 ft. deep which the bank-building machine itself has formed. Two chains with ⅓-cu. yd. buckets pick the material up from this feeder trench, and drop it into a hopper which discharges on to a 36-in. belt running upward at a gradient of 20° with a speed of 300 ft. per min. From this belt, the material falls upon a second belt which travels along a boom and is reversible. These two qualities combine to permit covering a width of 125 ft. from one position. The maximum dumping height is 50 ft. The theoretical capacity is 1 000 cu. yd. per hour, corresponding to twice 25 bucket discharges per min.; that is, there were two bucket ladders, each capable of discharging 25 buckets per min. The actual quantities placed in an average 18-hour day were from 6 000 to 7 500 cu. yd. This output is governed more by the efficiency of the transport system than by the machine's performance, and could be doubled under favorable circumstances.

It has been mentioned before that, to shorten interruptions due to repairs and reduce the required stock of spare parts, units of the different machines were made interchangeable. This idea was applied not only to mechanical parts such as excavator buckets, chain links, etc., but also to the electric equipment. Thus, the bank-building machines were equipped with the same types of motors, as far as feasible, as the multiple-bucket excavators. The two bucket chains and the upper conveyor belt were driven by 50-kw. units each and the lower belt by a 22-kw. motor. Besides these, there were the drives for the two winches for raising and lowering the bucket chains, a drive moving the lower belt along the boom, the advancing drive, and drives for the two pumps which were in readiness to supply water for sprinkling the fill if required.

Comparison with Other Methods.—Having described the machinery introduced, it may be well to compare its performance with other methods of constructing embankments. The time-honored method of building up the fill by dumping earth from tracks which are jacked up by a gang of men as the work progresses, was out of the question. The red marl is an excellent material as regards water-tightness, inasmuch as no more seepage than 0.1 gal. per sq. ft. per hour has been observed, but it is rather easily saturated with water, and it then tends to flow and slump together. Under these circumstances the track work, which plays an important part in this method, in any case, would have made the undertaking very expensive; the frequent rains would have stopped the work for days; and progress would have been very slow.

Two reasons mainly argue against the hydraulic-fill method: First, the quantity of water required would have to be carried from the river in a pipe line several miles long; and, second, a reason which in itself was sufficient to make the contractors abandon the idea, the material was wet enough as it was, in fact, part of it had to be excluded on account of its saturation and corresponding lack of stability which would have imperiled the embankments. Moreover, this method requires auxiliary equipment, such as drag-lines for building the shoulders, derricks, flumes, booster pumps, etc., and there would have been extensive auxiliary operations.

Dumping the material from trestles would have been another method; but apart from the great quantities of imported timber required for 15 miles of embankment, and the necessity of raising, not only the earth, but also the dead load of the locomotives and cars, to high elevations, there is the prejudice against leaving trestle bents in the completed fill.

General Advantages of New Method.—It is believed that the method adopted combines the advantages of the other systems without suffering from their drawbacks. Of all applicable methods, it is the one which does not require equipment of any description to travel or operate on new fill; the bank-building machine was the only unit used, and it traveled on the firm undisturbed ground at the land side of the embankment.

The two principal requirements for a structure used for impounding water are stability and impermeability. Apart from intelligent design of the cross-section, these requirements can be fulfilled by providing proper distribution of the material according to its water-tightness and weight, proper compacting of the fill, and avoidance of planes of weakness.

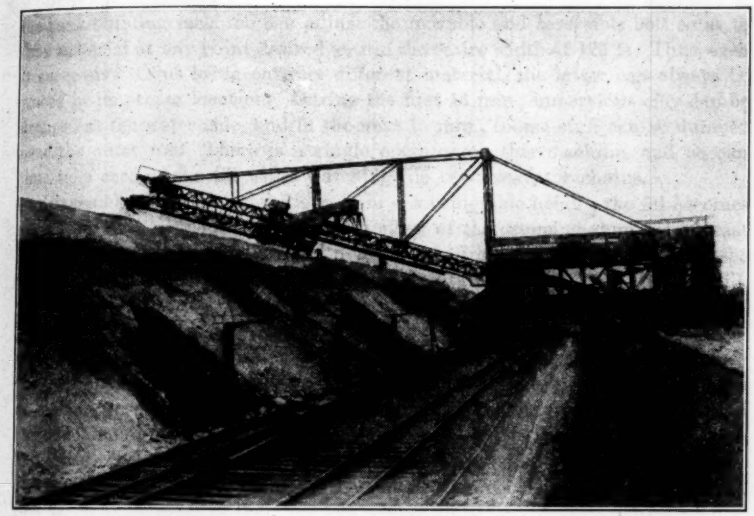


FIG. 6.—VIEW OF BANK-BUILDING MACHINE, SHANNON POWER DEVELOPMENT.

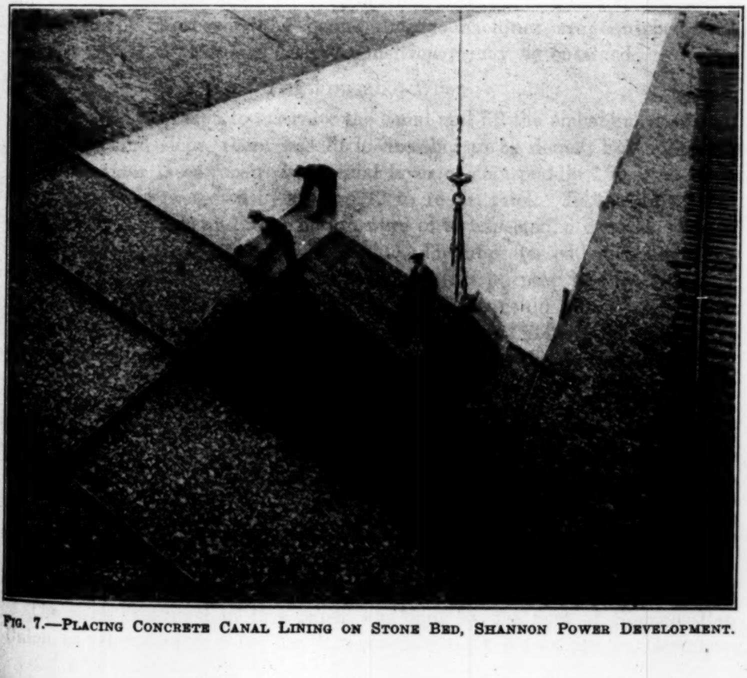


FIG. 7.—PLACING CONCRETE CANAL LINING ON STONE BED, SHANNON POWER DEVELOPMENT.

By pressing a contact button on the switchboard at his side, the operator of the bank-building machine can adjust the movable and reversible belt so as to drop material at any point desired within the entire width of 125 ft. Thus, even if successive trains bring entirely different material, the latter can always be placed in its proper location. During the first 15 min., impervious clay can be dropped at the water side, and in the next 15 min., coarse stuff can be dumped near the outer toe. There is a single operator on this machine, and he can thus take care of the fill while watching his two excavator chains.

Inasmuch as the material drops from a considerable height, the fill becomes very dense and compact. The tamping effect of the dropping stones is at least equivalent to that of a roller, and under some circumstances its effect will be greater. Moreover, there is the absolute certainty that every inch of fill, vertical and horizontal, has been compacted.

Rolled fills have a tendency to show what may be called "pancake" structure. Between well-compacted layers there occur looser strata which offer less obstruction to seepage. In the method described herein horizontal planes do not occur. When the lower conveyor belt is kept in a fixed position and the machine travels parallel to the canal axis, a longitudinal pile of earth is formed. Piles of, say, 5 ft. are built up side by side, and then the valleys are filled and new ridges formed. The whole dam consists of diamond-shaped prisms, so that, even if their contact planes did operate as weakness planes, the percolating water would have to climb one ridge after the other. Actually, the new material has been observed to pierce the old, and thus it forms a good bond. Moreover, the bank-building machines are equipped with sprinkler pipes, so that any desired consistency may be obtained.

The Finishing Work

Since care was taken to excavate the canal and fill the embankments to the proper level and slope, there was little dressing to be done. For additional safety the water faces received a special layer of clay puddle 1 to 2 ft. thick, which again was protected by rip-rap 12 to 16 in. thick. In the zone where water-level variations and wave action were to be expected, a concrete slab, 4¾ in. thick, was added as a further precaution. The unprotected banks and crown were re-soiled with the humus laid aside for this purpose in the beginning. For part of this work the bank-building machine could again be utilized; but part of it was done with the aid of swivel cranes. The latter also served for placing the rip-rap and concrete. Fig. 7 illustrates these operations.

Conclusion

A new method for handling large embankment building contracts has been explained. It is especially adapted to construction work involving great longitudinal extension. While it is mainly intended for handling lighter earths, it has also proved economical under severe conditions, in wet and viscous soil, and in a climate unfavorable to outdoor construction work. This fact is illustrated by the low price at which the work could be done; 1 cu. yd. of excavation and fill, including haulage, costs little more than 20 cents.

The work described in this paper was done by the firm of Siemens-Bau Union, as sub-contractors for the Siemens-Schuckert Works, Berlin, Germany.

DISCUSSION

L. J. Bevan,[4] M. Am. Soc. C. E. (by letter).—This paper presents in excellent form an interesting description of a brave project. From a power point of view Ireland is in an undeveloped condition similar to that of the United States about 1900. It has a total installation of 27 000 kw., with an urban population of 1 000 000, or 0.027 kw. per inhabitant. The annual consumption of 40 000 000 kw-hr. in 1920 when the Shannon Development was being proposed was 40 kw-hr. per inhabitant, as compared with 71 kw-hr. for all Europe and 472 kw-hr. for the United States. Economically, the Shannon Project should do much for the prosperity of Ireland.

This was referred to as a brave project primarily because of the large amount of power contemplated—500 000 000 kw-hr.—from 37 in. of rainfall on 4 000 sq. miles of drainage area, through a 100-ft. fall. At 80% efficiency from water to bus-bar 530 cu. ft. of water are required per kilowatt-hour. The output contemplated from the first development is 150 000 000 kw-hr., which corresponds to an average annual flow of 2 520 cu. ft. per sec., or a run-off-rainfall ratio of 23 per cent. This is reasonably to be expected.

The ultimate development, 500 000 000 kw-hr., similarly requires an average annual flow of 8 520 sec-ft., or a run-off-rainfall ratio of 77 per cent. Such a run-off ratio is so large that a further elaboration of this phase of the project would be of intense interest.

A. R. C. Markl,[5] Jun. Am. Soc. C. E. (by letter).—The general description of the development and its basis was curtailed by the writer to a minimum, so as to provide more space for the explanation of a novel method of handling earthwork, which formed the principal object of the paper. Mr. Bevan's questions demand an elaboration which would seem to be beyond its original purpose. Perhaps the following figures may serve as a reply.

In the 4 011-sq. mile area from which the Shannon River draws its water, an average annual rainfall of 37.25 in. obtains, corresponding to a total of 347 436 000 000 cu. ft. per year. Due to a signal lack of forests and the rockiness of the ground, 75 to 78% of this amount, that is, 270 156 000 000 cu. ft. per year, or 8 475 sec-ft., flow off. With a head varying between 93.5 and 110.7 ft., dependent on the tides, which travel up the tail-race, and an estimated rate of 584.8 cu. ft. per kw-hr., this produces 462 000 000 kw-hr. at the bus-bars.

The maximum flow occurs during the winter, while in the months from May to September rarely much more than 3 000 cu. ft. per sec. are carried by the river. This distribution, of course, is fortunate inasmuch as it produces a coincidence of supply and demand. An effective storage capacity of 20 235 000 000 cu. ft., which serves to achieve a complete balance of the water household, is obtained by providing for water-level variations of between 4.4 and 12.0 ft. at the three principal lakes through which the Shannon River flows. These lakes, Loughs Allen, Ree, and Derg, cover a total area of 101 sq. miles.

---

[4] Cons. Engr., New York, N. Y.

[5] Now Assoc. M. Am. Soc. C. E.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1776

PLASTIC FLOW IN CONCRETE ARCHES[1]

By Lorenz G. Straub,[2] Jun. Am. Soc. C. E.

With Discussion by Messrs. B. F. Jakobsen, C. A. P. Turner, Edward Godfrey, J. H. Griffith, Charles W. Comstock, A. A. Eremin, J. Melan, Fredrik Vogt and Herbert J. Gilkey, E. Probst, Paul Bauman, G. C. Staehle, A. Floris, and Lorenz G. Straub.

Synopsis

Many materials of construction, especially concrete during the first few months after fabrication, do not follow the laws upon which the theory of elasticity in its many forms is based. Yet this theory has been persistently used and elaborated for indeterminate structures, regardless of the material. Such factors as the yield of a stressed body with time, often called "hysteresis", and the variation of the ratio of stress to elastic strain, when at all accounted for in a structural analysis, have generally been analyzed in some more or less unscientific method of guess. For concrete arches, especially in Europe, construction is frequently furthered, that is, by the introduction of temporary hinges, hydraulic jacks, etc., in such a manner that these indeterminate factors can be controlled during the construction and early life of the structure.

Obviously, the ordinary theory of elasticity is entirely inadequate for studying stresses in a statically indeterminate structure made of a material which, when under load, yields with time as well as stress and does not obey Hooke's law for elastic deformation. To fill this need, methods of analysis have been developed by the writer based on experiment. These methods provide a much more general statement of the displacements and deformations

---

[1] This paper was developed from a phase of an unpublished investigation made by the writer while a "Fellow" in Civil Engineering at the University of Illinois, 1925–27. The title of the manuscript is "Neglected Factors in the Analysis of Stresses in Concrete Arches." Published in January, 1930, *Proceedings*.

[2] Associate Prof. of Hydraulics, Univ. of Minnesota, Minneapolis, Minn.

in structures than is given by the ordinary theory of elasticity because they fit a greater number of properties of materials. Further, they take the exact form of the usual theory of elasticity when plastic deformation is not considered and Hooke's law is assumed to apply. For convenience, the proposed methods of analysis have been termed the "theory of plasticity" and the "proposed theory of elasticity". Although these theories were developed in particular to investigate changes and variations of the stressed condition of concrete arches due to plastic flow, they are of general significance in structural design.

These investigations show definitely that one may be led astray in drawing conclusions regarding the stressed condition of an arch, even if based upon precise and accurate measurements of deformations. The studies indicate that it is even possible to find observed deformations to check the results of analysis by the ordinary elastic theory and yet the resulting conclusions might be wrong. Such misconceptions are readily verified by the procedures herein proposed.

Statements that a locally overstressed point in a structure relieves itself as a result of plastic flow, are shown to be unwarranted and misleading. A much clearer conception is given with regard to shrinkage stresses, temperature stresses, effects resulting from movement of arch abutments, etc. Valuable inferences may be drawn as to the proper interval before removing the arch centering.

NOMENCLATURE

Ordinarily, the nomenclature for arches is quite involved. When, in addition, such factors as plastic flow and variable stress-strain relations are considered, necessarily it becomes very cumbersome. Hence, some duplication has been used, but it has allowed simplification, and should not lead to confusion. In most instances, H_0 has been used to designate the neutral point horizontal thrust due to all causes, that is, superimposed load, dead load, rib-shortening, etc.; in a few cases, it refers to the thrust resulting only from dead and live loads.

As much as practicable of the ordinary American nomenclature for arches has been used. Simplifications, or rather condensations, and additions have also been adopted to make cumbersome algebraic expressions easy to handle. As an example, the symbol, V, has been used to replace a frequently recurring algebraic expression, much the same as the ordinary substitution of I, for the moment of inertia of a section.

Subscripts denote reference points and causes; for example, $H_{0\text{-}RS}$ indicates the horizontal thrust considered to act through the neutral point, O, of an arch, due to rib-shortening under load. Prime marks (superscripts) over a letter denoting a variable distance indicate a definite distance to a particular point or line. Thus, while z indicates the distance from the centroidal axis of a section to any point in the plane of the section, z' refers to the distance from the centroidal axis of the section to the line of action of the thrust acting upon the section.

In general, distances are considered positive when measured upward or to the right. Similarly, external force components having their sense upward or to the right are positive.

Thrusts acting within a member are considered positive when they cause tension. Shear at a section in a horizontal or inclined member is positive when the section resists impending motion upward at the left portion of the structure. In general, moments in the rib of an arch are called positive when they increase the compression at the extrados and decrease it at the intrados.

Angles are measured positive in a counter-clockwise direction from the initial line.

Notation.—The following nomenclature is common to the analyses presented. The less common terms are not included, but will be defined as they are introduced into the discussion.

$d\xi$ = the shortening along the arch axis of a differential length of the rib.

$d\tau$ = the transverse or sliding movement within a differential length.

$d\phi$ = the angle change within a differential length.

ds = a differential length of a structure measured along the centroidal axis.

$\Delta t°$ = the total temperature change, in degrees Fahrenheit.

α = the angle between a tangent to the arch axis and the x-axis of co-ordinates.

β = the angle which the quasi-horizontal x-axis, for an unsymmetrical arch, makes with the horizontal.

θ = the change in slope of the tangent at a point on a structure.

γ = the total unit of shrinkage of a given concrete at a particular time.

$\delta_1, \delta_2, \delta_n$ = respectively, the displacement of points in a structure.

δ_{xR}, δ_{xL} = respectively, the permanent displacements of the right and left springings of an arch in the direction of the x-axis of co-ordinates.

δ_{yR}, δ_{yL} = respectively, the permanent displacements of the right and left springings of an arch in the direction of the y-axis of co-ordinates.

δ_{rR}, δ_{rL} = respectively, the permanent rotational displacements of the right and left springings of an arch.

ϵ = the unit elastic deformation.

ϵ_z = the unit elastic deformation at a distance, z, from the centroidal axis.

ϵ_u = the unit elastic deformation at a distance, u, from the center of elastic rotation.

ϵ_u' = the unit elastic deformation along the centroidal axis.

μ = the coefficient of expansion per degree Fahrenheit.

ρ = the unit of plastic deformation.

ρ_v = the unit of plastic deformation at a distance, v, from the center of plastic rotation.

σ = unit stress.

σ_z = unit stress at a distance, z, from the centroidal axis.

σ_u = unit stress at a distance, u, from the center of elastic rotation.

b = the width of a section.

c = the distance from the centroidal axis of a section to the outer fiber.

c_0 = the distance from the centroidal axis to the outer fiber for a normal section at the crown of an arch.

d = the thickness of a normal section.

d_0 = the thickness of an arch at the crown.

E = Young's modulus of elasticity.

f = rise of arch from springing to crown.

f', f'', f''', these terms have been used freely to designate functions and constant factors in terms, especially where only relative values are of importance.

H_0 = the reaction at the center of co-ordinates considered to act in line with the x-axis at the end of a rigid arm fixed to the arch at the springing.

$H_{0\text{-}DL}$ = the horizontal thrust considered to act through the neutral point due to the dead load, that is, neglecting all effects of rib-shortening, temperature change, or shrinkage.

$H_{0\text{-}RS}$ = the horizontal thrust considered to act through the neutral point as a result of rib-shortening due to the compression of the arch.

H_R = the horizontal reaction in a symmetrical arch, or quasi-horizontal reaction in an unsymmetrical arch, at the right support when the arch is considered as a simple beam.

$I = \frac{2}{3} bc^3$ = the moment of inertia of a normal section according to the ordinary theory of elasticity.

K = a parameter in the exponential equation defining plastic deformation.

k = a parameter in the exponential equation defining elastic deformation.

l = the span of the arch measured parallel to the x-axis.

l' = the vertical distance between springings of the arch measured parallel to the y-axis.

Δl = increment or change in l, due to deformation of the arch.

$\Delta l'$ = increment in l', due to deformation.

Δl_{RS}, $\Delta l'_{RS}$ = increments, respectively, in l and l' due to rib-shortening in the arch.

m = an exponent of stress determined from experiment and defining the rate of elastic deformation with change in stress. The number, m, may also appear in equations as a coefficient.

M = the bending moment.

M_0 = the bending moment on a lever arm extending from the springing to the center of co-ordinates.

M_x = the bending moment at a section of the arch having the abscissa, x.

M_d = the bending moment caused by statically determinate forces.

M_i = the bending moment caused by statically indeterminate forces.

$M_{d\text{-}LL}$, $M_{d\text{-}DL}$
$M_{i\text{-}LL}$, $M_{i\text{-}DL}$ = respectively, the statically determinate moments due to live load and dead load on a structure, and the statically indeterminate moments due to live load and dead load on the structure.

N = the normal component of the resultant thrust acting on a radial section of the arch.

N_x, T_x, S_x = respectively, the normal thrust, resultant thrust, and shear at a section of the arch having the abscissa, x.

p = an exponent of stress determined from experiment and defining the rate of plastic deformation with a change in stress. The number, p, may also appear as a coefficient in equations which have been differentiated or integrated.

q = an exponent of time determined from experiment and defining the rate of plastic deformation with duration of loading.

$P_1, P_2, \ldots P_n$ = concentrated loads.

$$Q = \left(b\,c^{\frac{1+2m}{m}} \times \frac{2m}{1+2m}\right).$$

$$S = \left(b\,c^{\frac{1+2p}{p}} \times \frac{2p}{1+2p}\right).$$

S = also, the shear component of the resultant thrust acting on a radial section of the arch.

s = the length of the arch axis.

T = the resultant thrust on a radial section of the arch.

t = the duration of a stressed condition, in days.

T_{RS}, N_{RS}, S_{RS}, and M_{RS} = the resultant thrust, normal thrust, shear, and moment at a section in an arch due to a horizontal rib-shortening thrust, $H_{0\text{-}RS}$.

$$U = b\,c^{\frac{1+m}{m}} \times \frac{m}{1+m}\left[\left(\frac{u'}{c}+1\right)^{\frac{1+m}{m}} - \left(\frac{u'}{c}-1\right)^{\frac{1+m}{m}}\right].$$

$$U' = \frac{m}{1+m}\left[\left(\frac{u'}{c}+1\right)^{\frac{1+m}{m}} - \left(\frac{u'}{c}-1\right)^{\frac{1+m}{m}}\right].$$

$$V = b\,c^{\frac{1+p}{p}} \times \frac{p}{1+p}\left[\left(\frac{v'}{c}+1\right)^{\frac{1+p}{p}} - \left(\frac{v'}{c}-1\right)^{\frac{1+p}{p}}\right].$$

$$V' = \frac{p}{1+p}\left[\left(\frac{v'}{c}+1\right)^{\frac{1+p}{p}} - \left(\frac{v'}{c}-1\right)^{\frac{1+p}{p}}\right].$$

V_0 = the vertical reaction at the center of co-ordinates, considered to act at the end of a rigid arm fixed to the arch at the springing.

u = the distance from the center of elastic deformation to a fiber in a section.

u' = the distance from the center of elastic deformation at a section to the centroidal axis.

v = the distance from the center of plastic deformation to a fiber in a section.

v' = the distance from the center of plastic deformation at a section to the centroidal axis.

$V_1, V_2, \ldots V_n$, and $H_1, H_2, \ldots H_n$ = the vertical and horizontal or quasi-horizontal components, respectively, of the forces, $P_1, P_2, \ldots P_n$.

V_R, V_L = the vertical reaction at the right and left supports, respectively, of an arch considered as a simple beam.

w = the magnitude of a uniformly distributed load per foot of length. (In a few instances, w is used to designate the distance from the center of combined elastic and plastic deformation to a fiber in a section.)

x, y = respectively, abscissa and ordinate of a point referred to a system of co-ordinates.
z = the distance of a fiber from the centroidal axis.
z' = the eccentricity of the line of thrust with respect to the arch axis.

I.—GENERAL STATIC AND GEOMETRIC RELATIONS FOR A DEFORMED ARCH

Fundamental Relations.—In considering the action of any structure under load, be the structure statically determinate or indeterminate, attention is given to three principal items. These are:

1.—Geometric Relations.—The relation of the angle changes and translations of the differential parts of the structure must be such that the geometric requirements imposed on the structure are fulfilled, for example, constant span length, etc.

2.—Static Equilibrium.—The sum of the components in any direction of all forces acting on the whole structure, or on any part of the structure, must be equal to zero; the sum of the moments acting on the whole, or on any part of, the structure must be equal to zero.

3.—Properties of the Material of Construction.—Deformations and stresses in differential parts of the structure must be so related as to satisfy the properties of the material, for example, the ratio of stress to strain should correspond to that obtained by experiment.

If a structure is statically determinate, the reactions resulting from a given condition of loading can be found without considering the properties of the material. However, even then, the internal stresses due to a given loading are dependent on the elastic properties of the material, and in some cases may involve the plastic properties or deformation with time as well as stress.

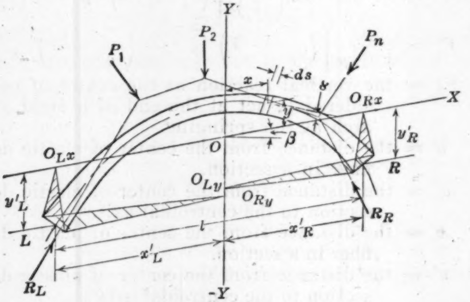


FIG. 1.

When a statically indeterminate structure is analyzed, all three of the aforementioned factors must be combined into equations before a satisfactory solution can be obtained.

Development of Geometric Relations.—Consider an arch of arbitrary shape (Fig. 1) subjected to any system of loading in its plane. Due to the assumed loads, P_1, P_2, P_n, there are the reactions, R_L and R_R, at the left and right ends, respectively.

After loading (Fig. 2), the arch assumes a new conformation dependent on the movement of the abutments, the bending, and the elongation and shortening of elemental lengths of the arch rib. The deformations may be due to any one or any combination of the following causes:

(1) Rotation or translation of the abutments.
(2) Temperature change.
(3) Shrinkage of materials composing the arch.
(4) Plastic flow of materials composing the arch.
(5) Elastic deformation due to stress.

Assume (Fig. 1) a center of co-ordinates, O, and co-ordinate axes, x and y. A rigid lever arm is assumed to be connected to each springing and extended to the x-axis and parallel to the y-axis. The length of the left lever arm is y'_L; that at the right is y'_R. Similarly, rigid lever arms of lengths, x'_L and x'_R, are connected to the left and right springing, respectively, and extended to the y-axis and parallel to the x-axis.

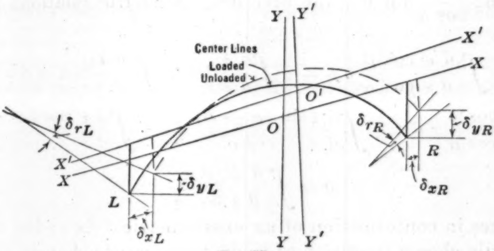


FIG. 2.

Consider a differential length of the arch rib, ds, the abscissa of which is x. The angle change within this differential length, due to the strained condition of the arch, is $d\,\phi$. The rate of angle change then is given by $\frac{d\,\phi}{d\,s}$.

The relative movement of O_{Lx} and O_{Rx} in the direction of the x-axis, due to the differential angle change, $\frac{d\,\phi}{d\,s}$, is equal to $\frac{d\,\phi}{d\,s}\,y\cos\beta\,d\,s$.

Denote the shortening of a differential length, $d\,s$, the abscissa of which is x, by $d\,\xi$. Hence, the shortening (or lengthening) of the differential length in the x-direction will be $\frac{d\,\xi}{d\,s}\cos\alpha\,d\,s$, in which, α is the angle the arch axis makes with the x-axis.

Considering the sliding or shear deformation perpendicular to the arch axis in a length, $d\,s$, to be $d\,\tau$, the differential movement in the x-direction is found to be $\frac{d\,\tau}{d\,s}\sin\alpha\,d\,s$.

As a result of these considerations, the equation for the relative movement of O_{Lx} and O_{Rx} along the x-axis is,

$$\Delta\,l = \delta_{xR} - \delta_{xL} = \int \frac{d\,\phi}{d\,s}\,y\cos\beta\,d\,s + \int \frac{d\,\xi}{d\,s}\cos\alpha\,d\,s$$
$$+ \int \frac{d\,\tau}{d\,s}\sin\alpha\,d\,s - (y'_L\,\delta_{rL} - y'_R\,\delta_{rR})\cos\beta \ldots\ldots\ldots\ldots (1)$$

Similarly, it may be shown that the relative movement of O_{Ly} with respect to O_{Ry} along the y-axis is,

$$\Delta l' = \delta_{yR} - \delta_{yL} = \int \frac{d\,\phi}{d\,s}\, x \cos\beta\, d\,s + \int \frac{d\,\xi}{d\,s} \sin(\alpha - \beta)\, d\,s$$

$$+ \int \frac{d\,\tau}{d\,s} \cos(\alpha - \beta)\, d\,s - (x'_L\, \delta_{rL} - x'_R\, \delta_{rR}) \cos\beta \ldots\ldots\ldots (2)$$

The rotation of the left springing with respect to the right springing is,

$$\Delta\, r = \delta_{rR} - \delta_{rL} = \int \frac{d\,\phi}{d\,s}\, d\,s \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Equations (1), (2), and (3) define the changed configurations of the arch due to all possible causes. These relations are geometrical and are independent of the properties of the material or conditions of equilibrium.

Considering that there is no translation or rotation of the ends of the arch, and substituting $\dfrac{dx}{\cos\alpha}$ for $d\,s$, the preceding geometric relations reduce to the forms,

$$\left.\begin{aligned}
0 &= \int \frac{d\,\phi}{d\,s} \frac{\cos\beta}{\cos\alpha}\, y\, d\,x + \int \frac{d\,\xi}{d\,s}\, d\,x + \int \frac{d\,\tau}{d\,s} \tan\alpha\, d\,x \\
0 &= \int \frac{d\,\phi}{d\,s} \frac{\cos\beta}{\cos\alpha}\, x\, d\,x + \int \frac{d\,\xi}{d\,s} \frac{\sin(\alpha - \beta)}{\cos\alpha}\, d\,x + \int \frac{d\,\tau}{d\,s} \frac{\cos(\alpha - \beta)}{\cos\alpha}\, d\,x \\
0 &= \int \frac{d\,\phi}{d\,s} \frac{d\,x}{\cos\alpha}
\end{aligned}\right\} \ldots (4)$$

The changes in conformation of an unsymmetrical fix-ended arch subjected to an arbitrarily chosen condition of loads, temperature change, shrinkage, etc., are thus defined by Equations (4).

For a symmetrical arch these equations may be written in the form:

$$\left.\begin{aligned}
0 &= \int \frac{d\,\phi}{d\,s} \frac{y\, d\,x}{\cos\alpha} + \int \frac{d\,\xi}{d\,s}\, d\,x + \int \frac{d\,\tau}{d\,s} \tan\alpha\, d\,s \\
0 &= \int \frac{d\,\phi}{d\,s} \frac{x\, d\,x}{\cos\alpha} + \int \frac{d\,\xi}{d\,s} \tan\alpha\, d\,x + \int \frac{d\,\tau}{d\,s}\, d\,x \\
0 &= \int \frac{d\,\phi}{d\,s} \frac{d\,x}{\cos\alpha}
\end{aligned}\right\} \ldots\ldots\ldots (5)$$

Relations Defining Static Equilibrium.—An unsymmetrical arch (Fig. 3) is subjected to an arbitrary system of loading, P_1, P_2, P_n, in the plane of the arch, with reactions, R_L and R_R, occurring at the left and right ends of the arch, respectively. Fig. 3 shows the arch under load and in its deformed state.

The arch has been made statically determinate with respect to the external forces and reactions in Fig. 4, while at the same time it is held in its deformed state by forces applied to the ends of rigid lever arms which are fixed to the left and right springings respectively, and terminate at the center of coordinates. The arch in its deformed state is held in equilibrium by a system of forces which may be divided into two classes, namely, (1) statically determinate forces and reactions, and (2) redundant forces.

The statically determinate forces and reactions are, respectively, the applied loads, P_1, P_2, P_n, and the reactions, V_L, V_R, and H_R. The reaction, H_R, is a quasi-horizontal force having its point of application at the right end of the arch and its direction parallel to the x-axis. The statically determinate forces may be considered as an independent system holding the structure in equilibrium. These are independent of internal stresses in the arch caused by deformations. Thus,

$$\Sigma H_R = \Sigma H_n$$
$$V_L + V_R = \Sigma V_n$$
$$\Sigma M \text{ (due to } P) = \Sigma M \text{ (due to reactions)}$$

On the other hand, the redundant forces are caused by restrained conditions of the ends of the arch and are directly caused by temperature changes, rib-shortening, movement of abutments, etc. The components of these forces must exist in pairs equal in magnitude and opposite in sense, for unless this is true the structure will not be maintained in a state of equilibrium. The redundant forces required to hold the structure in the conformation assumed are arbitrarily divided into H_0, V_0, and M_0-components at the center of the co-ordinate system. The H_0-components act along the x-axis, whereas the V_0-components are directed along the y-axis. These components may be considered to act in pairs of equal and opposite forces, one set acting upon the end of each lever arm.

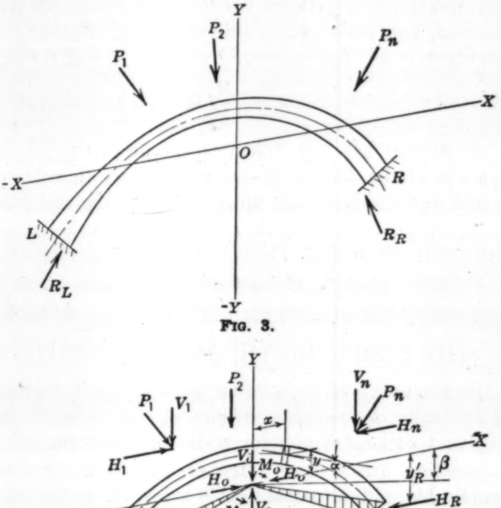


FIG. 3.

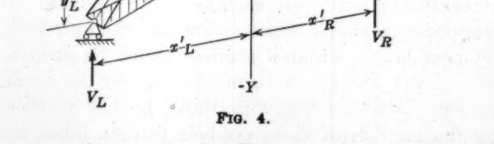


FIG. 4.

It is well to remember, comparing Figs. 3 and 4, that the resultant of the components, H_0, V_0, and M_0, at the end of the left lever arm, combined with the vertical reaction, V_L, is the reaction, R_L. Similarly, the resultant of the components considered to act at the end of the right lever arm and the reactions, V_R and H_R, form the reaction, R_R.

Consider, now, a normal section through the arch axis, having the abscissa, x. The stress functions due to the statically determinate forces will be denoted by a subscript, d; functions due to redundant forces will be denoted by a subscript, i. Hence, the moment at a section of the arch having the abscissa, x, will be given by $M_x = M_{xd} + M_{xi}$. Thus, considering the sense of the redundant forces on the left lever arm, the moment, normal thrust, and shear, respectively, are found to be:

$$\left.\begin{array}{l} M_x = M_{xd} + M_0 + V_0\, x \times \cos\beta - H_0\, y \times \cos\beta \\ N_x = N_{xd} + H_0 \times \cos\alpha + V_0 \times \sin(\alpha - \beta) \\ S_x = S_{xd} + H_0 \times \sin\alpha + V_0 \times \cos(\alpha - \beta) \end{array}\right\} \ldots\ldots\ldots (6)$$

For the case of a symmetrical arch referred to a rectangular system of co-ordinates, Equations (6) become:

$$\left.\begin{array}{l} M_x = M_{xd} + M_0 + V_0\, x - H_0\, y \\ N_x = N_{xd} + H_0 \times \cos\alpha + V_0 \times \sin\alpha \\ S_x = S_{xd} + H_0 \times \sin\alpha + V_0 \times \cos\alpha \end{array}\right\} \ldots\ldots\ldots\ldots\ldots (7)$$

Equations (6) for the unsymmetrical arch and Equations (7) for the symmetrical arch, respectively, are simply equations of statics defining the magnitude, line of action, and sense of the line of thrust at a section of the arch the abscissa of which is x. The eccentricity of the line of thrust is,

$$z' = \frac{M_x}{N_x} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Thus far, no account has been taken of the properties of the material of construction in any of the derived equations. It may then be concluded that the geometric and static relations set up in the foregoing analysis are not open to question on the ground that they do not take into consideration the correct elastic properties of the material. The equations are true regardless of whether the material is elastic, plastic, or whether it possesses a composite of both properties.

II.—THE PROPOSED THEORY OF ELASTICITY

While in the computation of bending moments and the action of outer forces on all statically determinate structures, the rules of statics may be applied directly and without doubt, for statically indeterminate structures of concrete, the question arises as to whether or not the ordinary theory of elasticity is applicable without considerable error.

It is appalling to consider how definitely Hooke's law is ingrained in all the structural engineer's artificial machinery for analysis. Indeed, he has started with members in simple compression and flexure; from their mathematical development he has grouped symbols which frequently appear in the same combination, and has given them such names as moment of inertia, elastic weight, etc. Not satisfied with these, he has further superimposed laws governing the relation of these symbols to each other, and has evolved such combinations as Maxwell's law of reciprocal deflections, the theorems of virtual work, methods of substitute beams, moment areas, slope deflections, and others. All these methods and theorems are developed assuming the variation of stress with strain to be governed by Hooke's law.

The assumption of a constant elastic modulus is correct for steel within the elastic limit, at least as far as designs subjected only to static loads are concerned. However, for concrete, this assumption is known to be incorrect and has resulted in much controversy. Many have doubted the applicability of the ordinary theory of elasticity to indeterminate concrete structures. To be sure, experiments are recorded indicating that the theory of elasticity gives results at least approximately correct for short-time loads. From a group of experiments performed on concrete frames,[3] M. Abe, M. Am. Soc. C. E., concludes that, "the elastic action of the frames under external load and the manner of stress distribution along members of the frame agree fairly well with the analysis given." (Dr. Abe's analysis was an application of a form of the ordinary theory of elasticity.[4]) Theoretical studies of the "Austrian Experiments on Arches"[5] indicates that the theory of elasticity, at least for loads within the working stresses of the material, gives reasonable results in a stress analysis for rubble masonry and brick arches as well as for concrete arches.

However, considering the degree of precision obtained by the ordinary theory of elasticity, it is unscientific, to say the least, not to determine what effect a variable modulus of elasticity has upon the moment and stress distribution within a structure. Of course, a theory based merely upon the properties of the material and not yet substantiated by at least a few scientific tests upon simple structures is not entirely ready for complete acceptance. Yet, no investigations are so unscientific and unreliable as those made upon built-up units, before a feasible theory of analysis has been developed. It is these considerations that have led the writer to develop what he believes to be a more flexible theory of elasticity, that is, one which accounts for a variable modulus of elasticity.

It is of some interest to note that Professor Melan (in studies to determine the modulus of elasticity of the concrete forming the "Stampf-Beton" arches tested by the Austrian Committee) found that the computed elastic modulus changed very greatly as the load increased, being less for the greater loads. His work was carried out by taking the actual vertical and horizontal displacements into account in an ordinary elastic analysis. Thus, one might conclude that the study of deformations in themselves have little meaning in the investigation of stresses in arches.

Basis for Proposed Theory.—After studying the stress-strain relation of considerable experimental data for concrete, the writer has chosen the equation,

$$\epsilon = k\,\sigma^{m} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

to define the relation. Bach[6] developed an equation similar to this except that instead of k he used the reciprocal of the initial modulus of elasticity. However, this expression did not prove as satisfactory to the writer and is less flexible in application. Bach and Schüle[7] through a large number of experiments found the exponent, m, in their equation to range between the limits of 1.10 and 1.16 for various mixes of concrete.

---

[3] *Bulletin No. 107,* Univ. of Illinois Eng. Experiment Station, p. 100.

[4] Assuming stress proportional to strain and no plastic deformations.

[5] Bericht des Gewölbe-Ausschusses, *Zeitschrift* des Oesterr. Ingenieur- und Architekten-Vereines, 1896.

[6] Bach and Bauman, "Elastizität und Festigkeit," Berlin, 1924.

For concrete which was made into tension and compression specimens at the same time it was found that the elastic modulus in tension[8] did not differ greatly from that in compression, loads being applied in steps for the first time. This relation does not necessarily follow in all cases, but seems to be true in general for loads applied for the first time.

Data obtained from typical experiments by Dr. E. Probst[9] have been used by the writer as a basis for analysis in the basic equation, and curves (not herein included) were found which fit the data exceedingly accurately. The data included were for concrete 70 days old and of a 1 : 2 : 4 mix made with 10.1% water, giving a "very plastic material." Using the English system of units, for compression, the data are defined by,

$$\varepsilon = 178 \times 10^{-9}\, \sigma^{1.105}$$

In tension, the elasticity of the specimens used is represented by,

$$\varepsilon = 192 \times 10^{-9}\, \sigma^{1.03}$$

Development of Proposed Theory of Elasticity for Flexure.—When analyzing a concrete structure, it is hopeless to get a theory that is altogether exact. However, close approximations may be made, resulting in a study which will approach the actual conditions. Concrete in tension will follow a slightly different law for the relation of stress to deformation from that followed in compression. However, the general exponential relation is close for both. If the permanent set be included in the deformation represented in the exponential equation, then the equation, based on Probst's experiments, is practically the same for tension and compression.

Based upon these considerations, a theory of analysis is developed which accounts for a variation of the modulus of elasticity with variation in stress.

Structures are grouped into two classes, namely, (1) those subjected to pure flexure; and (2) those subjected to combined flexure and compression.

Development of the Flexure Equations According to the Proposed Theory of Elasticity.—The relation between elastic deformation and stress is given by,

$$\varepsilon = k\, \sigma^{m} \quad \text{.............................(9)}$$

Fig. 5 indicates a beam loaded in an arbitrary manner. Consider a differential length, $d\,s$. The section of the beam (Fig. 6(a)) is assumed rectangular, having a width, b, and depth, d, or $2c$.

Let, $A = b\,d =$ the area of the section. By the principles of statics,

$$M = \int \sigma_z\, z\, d\,A$$

or, considering σ_z to be independent of b,

$$M = \int_{-c}^{c} b\, \sigma_z\, z\, d\,z = 2\,b \int_{0}^{c} \sigma_z\, z\, d\,z \quad \text{...............(10)}$$

---

[7] *Zeitschrift* des Vereines deutscher Ingenieure, 1898.

[8] *Mitteilungen* über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, Heft 227, pp. 49–51.

[9] "Einfluss der Armature und der Risse im Beton auf die Tragsicherheit," E. Probst, *Mitteilungen* aus dem Königlichen Materialprüfungsamt zu Gross-Lichterfelde West, Ergänzungsheft I, pp. 8-14, Verlg. von Julius Springer, 1907.

A transformation of Equation (9) shows that,

$$\sigma_z = \left(\frac{\varepsilon_z}{k}\right)^{\frac{1}{m}} \ldots\ldots\ldots\ldots (11)$$

From Fig. 6(*b*), it may be seen that, assuming conservation of plane sections during bending,

$$\varepsilon_z = z \frac{d\,\phi_E}{d\,s} \ldots\ldots\ldots\ldots (12)$$

Combining Equations (10), (11), and (12), and rewriting,

$$M = \frac{2\,b}{k^{\frac{1}{m}}} \left(\frac{d\,\phi_E}{d\,s}\right)^{\frac{1}{m}} \int_0^c z^{\frac{1+m}{m}}\, d\,z$$

from which, when integrated and transformed,

$$\frac{d\,\phi_E}{d\,s} = \frac{k\,M^m}{\left[\dfrac{2\,m}{1+2\,m}\, b\, c^{\frac{1+2\,m}{m}}\right]^m}$$

or,

$$\frac{d\,\phi_E}{d\,s} = k \left(\frac{M}{Q}\right)^m \ldots\ldots\ldots\ldots (13)$$

in which,

$$Q = \frac{2\,m}{1+2\,m}\, b\, c^{\frac{1+2\,m}{m}} \ldots\ldots\ldots\ldots (14)$$

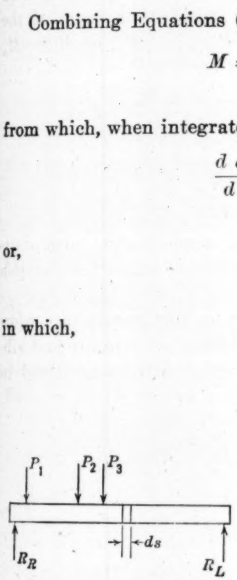
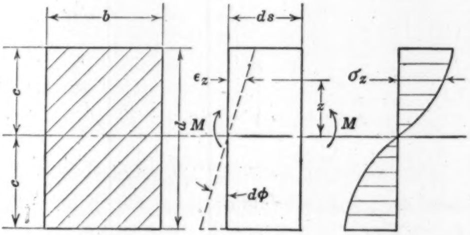


FIG. 5.



(*a*) (*b*) (*c*)

FIG. 6.

Equation (13) defines the rate of angle change at a section of a rectangular beam in terms of the bending moment, dimensions of the section, and elastic properties of the material.

Special Case in Which m = 1.—Assuming that $m = 1$, as in the ordinary theory of elasticity,

$$Q = \frac{2}{3}\, b\, c^3 = \frac{1}{12}\, b\, d^3 = I$$

and,

$$\frac{d\,\phi_E}{d\,s} = \frac{M}{E\,I}$$[10]

---

[10] The value, E, is here assumed to be equal to $\frac{1}{k}$. As a matter of fact, k should be chosen from experimental data and a value differing somewhat from this will better suit the data.

DERIVATION OF PROPOSED THEORY OF ELASTICITY FOR ARCHES

General Relations.—A method of analysis in which account is taken of the fact that strain is an exponential function of the stress becomes somewhat complicated because the law of superimposition of stresses and strains does not apply; that is, in order to reach the final stress and strain condition, one may not combine the results obtained by considering separately the effects of individual components of a force system. For example, consider a member in simple flexure. The total bending moment, M, is equal to the sum of the bending moments due to the individual loads, P_1, P_2, ..., P_n, or $M = M_1 + M_2 \ldots. + M_n$. The rate of angle change at any section is,

$$\frac{d\ \phi_E}{d\ s} = k\left(\frac{M}{Q}\right)^m = k\ \frac{(M_1 + M_2 + \ \ldots. + M_n)^m}{Q^m}$$

This equation can not be simplified to a form,

$$\frac{d\ \phi_E}{d\ s} = \frac{M_1}{E\ I} + \frac{M_2}{E\ I} + \ldots. + \frac{M_n}{E\ I}$$

as in the ordinary theory of elasticity. As a result, when dealing with statically indeterminate structures, it is necessary to handle the entire force system in a single analysis.

Since shear deformations have only slight effect on the change in conformation of a loaded arch rib in comparison to the effects of moment and rib-shortening deformations, the deformations due to shear will be neglected in the analysis.

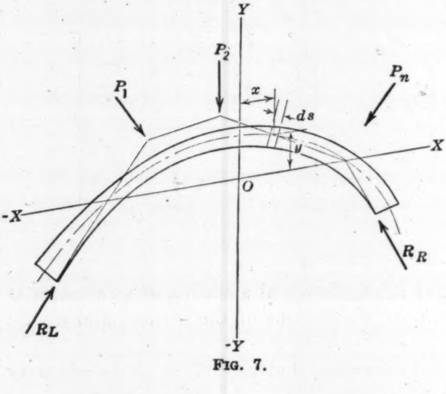


FIG. 7.

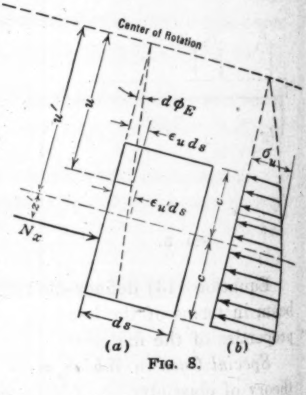


FIG. 8.

Derivation of Equations.—Consider the differential length, $d\ s$, of an arch rib (Fig. 7) loaded in an arbitrary manner. The rib is assumed to be rectangular in section. The width of the section is b, and the depth is d or $2c$.

In Fig. 8(a) the line of action of the normal thrust is N_x, and the deformed segment is $d\ s$. The distribution of stresses across the section is indicated in Fig. 8(b).

The relation of strain to stress is given by,

$$\varepsilon_u = k\ \sigma_u{}^m \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

By geometry (Fig. 8(*a*)),

$$\varepsilon_u = u\,\frac{d\,\phi_E}{d\,s} \qquad (16)$$

Subtracting Equation (16) from Equation (15) and re-arranging terms,

$$\sigma_u = \left(\frac{u}{k} \times \frac{d\,\phi_E}{d\,s}\right)^{\frac{1}{m}} \qquad (17)$$

The following relations must be true in order to maintain static equilibrium:

$$N_x\,(u' + z') = \int \sigma_u\, u\, d\,A \qquad (18)$$

$$N_x = \int \sigma_u\, d\,A \qquad (19)$$

By substituting $b \times d\,u$ for $d\,A$, the differential area of the section, and the value found in Equation (17), for σ_u, Equation (19) becomes,

$$N_x\,(u' + z') = \frac{b}{k^{\frac{1}{m}}} \times \left(\frac{d\,\phi_E}{d\,s}\right)^{\frac{1}{m}} \int_{(u'-c)}^{(u'+c)} u^{\frac{1+m}{m}}\, d\,u$$

from which,

$$N_x(u' + z') = \frac{m}{1 + 2m} \times \frac{b}{k^{\frac{1}{m}}} \times \left(\frac{d\,\phi_E}{d\,s}\right)^{\frac{1}{m}} \left[(u' + c)^{\frac{1+2m}{m}} - (u' - c)^{\frac{1+2m}{m}}\right] \qquad (20)$$

Similarly, by making substitutions for $d\,A$ and σ_u in Equation (19), and integrating between the same limits,

$$N_x = \frac{m}{1 + m} \times \frac{b}{k^{\frac{1}{m}}} \times \left(\frac{d\,\phi_E}{d\,s}\right)^{\frac{1}{m}} \left[(u' + c)^{\frac{1+m}{m}} - (u' - c)^{\frac{1+m}{m}}\right] \qquad (21)$$

Equation (20) divided by Equation (21) gives,

$$(u' + z') = \frac{1 + m}{1 + 2m} \times \frac{\left[(u' + c)^{\frac{1+2m}{m}} - (u' - c)^{\frac{1+2m}{m}}\right]}{\left[(u' + c)^{\frac{1+m}{m}} - (u' - c)^{\frac{1+m}{m}}\right]} \qquad (22)$$

Transposing u' to the second member of the equation and dividing both members by c,

$$\frac{z'}{c} = \frac{1 + m}{1 + 2\,m} \times \frac{\left[\left(\frac{u'}{c} + 1\right)^{\frac{1+2m}{m}} - \left(\frac{u'}{c} - 1\right)^{\frac{1+2m}{m}}\right]}{\left[\left(\frac{u'}{c} + 1\right)^{\frac{1+m}{m}} - \left(\frac{u'}{c} - 1\right)^{\frac{1+m}{m}}\right]} - \frac{u'}{c} \qquad (23)$$

From Equation (21),

$$\frac{d\,\phi_E}{d\,s} = k\,\frac{N_x^{\,m}}{\left\{b\,c^{\frac{1+m}{m}}\,\frac{m}{1 + m}\left[\left(\frac{u'}{c} + 1\right)^{\frac{1+m}{m}} - \left(\frac{u'}{c} - 1\right)^{\frac{1+m}{m}}\right]\right\}^{m}}$$

or,

$$\frac{d\,\phi_E}{d\,s} = k\left(\frac{N_x}{U}\right)^{m} \qquad (24)$$

in which,

$$U = \left\{ b\,c^{\frac{(1+m)}{m}}\,\frac{m}{1 + m}\left[\left(\frac{u'}{c} + 1\right)^{\frac{1+m}{m}} - \left(\frac{u'}{c} - 1\right)^{\frac{1+m}{m}}\right]\right\} \qquad (25)$$

The rate of angle change in the arch at a radial section the abscissa of which is x, is given by Equation (24) in terms of the dimensions of the section, elastic properties of the material, magnitude of the normal thrust, and distance between the center of rotation and center of gravity of the section. Equation (23) shows the relation between the eccentricity of the thrust and the distance to the center of rotation. Thus, Equations (23) and (24) together fully define the angle change in terms of the moment, thrust, and properties of the section.

The rib-shortening, or the shortening at the gravity axis, of an elemental length, $d\,s$, is,

$$d\,\xi_E = \varepsilon_{u'}\,d\,s$$

but by geometry,

$$\varepsilon_{u'} = u'\,\frac{d\,\phi_E}{d\,s}$$

hence,

$$\frac{d\,\xi_E}{d\,s} = u'\,\frac{d\,\phi_E}{d\,s} \qquad \ldots\ldots (26)$$

Substituting for $\frac{d\,\phi_E}{d\,s}$, the value found in Equation (24),

$$\frac{d\,\xi_E}{d\,s} = u'\,k\left(\frac{N_x}{U}\right)^m \qquad \ldots\ldots (27)$$

Neglecting shear deformations and substituting values for $\frac{d\,\phi}{d\,s}$ and $\frac{d\,\xi}{d\,s}$, respectively, from Equations (24) and (27) into the geometric Equations (5),

$$\left.\begin{aligned} 0 &= \int k\left(\frac{N}{U}\right)^m \frac{y\,d\,x}{\cos\alpha} + \int u'\,k\left(\frac{N}{U}\right)^m d\,x \\ 0 &= \int k\left(\frac{N}{U}\right)^m \frac{x\,d\,x}{\cos\alpha} + \int u'\,k\left(\frac{N}{U}\right)^m \tan\alpha\,d\,x \\ 0 &= \int k\left(\frac{N}{U}\right)^m \frac{d\,x}{\cos\alpha} \end{aligned}\right\} \ldots\ldots (28)$$

By substituting $\frac{M}{z'}$ for N in Equations (28), and replacing the definite integrals by summation of finite increments, the equations become,

$$\left.\begin{aligned} 0 &= k\sum\left(\frac{M}{z'\,U}\right)^m \frac{y\,\Delta\,x}{\cos\alpha} + k\sum u'\left(\frac{M}{z'\,U}\right)^m \Delta\,x \\ 0 &= k\sum\left(\frac{M}{z'\,U}\right)^m \frac{x\,\Delta\,x}{\cos\alpha} + k\sum u'\left(\frac{M}{z'\,U}\right)^m \tan\alpha\,\Delta\,x \\ 0 &= k\sum\left(\frac{M}{z'\,U}\right)^m \frac{\Delta\,x}{\cos\alpha} \end{aligned}\right\} \ldots (29)$$

Special Case, $m = 1$.—Equations (29) defining the proposed theory of elasticity for arches readily reduce to those of the ordinary theory of elasticity.

Solution of Proposed Elastic Equations.—Equations (29) fully define the elastic equilibrium of the arch, that is, its change in shape. However, a solution of these equations is not as simple as that when the ordinary straight-line, stress-strain variation is assumed. Angle changes in elementary radial segments of the arch are dependent not only on the bending moment, but also on the eccentricity of the force producing the moment. No particular choice of the center of co-ordinates seems to aid materially in the analysis, because the deformations due to separate forces acting independently may not be superimposed to obtain the deformation caused by the combined system of forces.

Assume, now, the value of m to be known from experiment. From Equation (23), values may then be computed showing the relation between the eccentricity, z', of the compressive force, and the distance, u', between the center of rotation and the gravity axis of the section. A more general relation is obtained by finding the relation between $\frac{z'}{c}$ and $\frac{u'}{c}$, because such a relation is directly applicable to a section of any thickness. Using constant values of m, curves have been plotted (Fig. 9) with $\frac{z'}{c}$ values as abscissas and $\frac{u'}{c}$ values as ordinates.

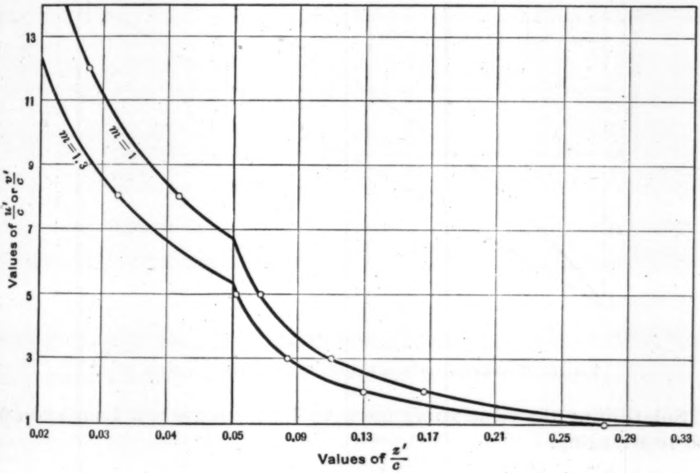


FIG. 9.—CURVES EXPRESSING RELATION BETWEEN $\frac{z'}{c}$ and $\frac{u'}{c}$.

From Equation (25),

$$U = b\, c^{\frac{1+m}{m}}\, U' \dots\dots\dots\dots\dots\dots\dots\dots (30)$$

in which,

$$U' = \frac{m}{1+m}\left[\left(\frac{u'}{c}+1\right)^{\frac{1+m}{m}} - \left(\frac{u'}{c}-1\right)^{\frac{1+m}{m}}\right] \dots\dots\dots (31)$$

A relation of $\frac{1}{U'}$ and $\frac{z'}{c}$ is obtained between Equations (23) and (31) by substituting the same value of $\frac{u'}{c}$ in both equations and solving, respectively, for the corresponding values of $\frac{1}{U'}$ and $\frac{z'}{c}$. Curves have been plotted (Fig. 10) with $\frac{z'}{c}$ values as abscissas and $\frac{1}{U'}$ values as ordinates. For $m = 1$, the curve is a straight line passing through the origin of co-ordinates. Hence, the expression, $\frac{1}{\frac{z'}{c} U'}$, is constant for all values of z'. Consequently, Equations (32), which have been derived from Equations (29), take the form of the ordinary theory of elasticity for $m = 1$.

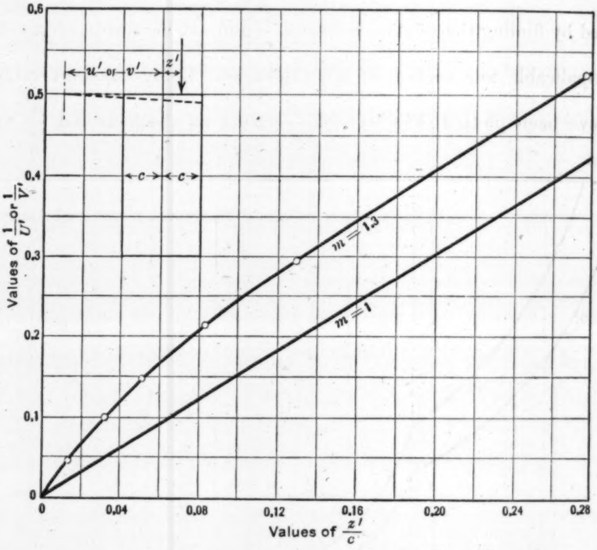


FIG. 10.—CURVES EXPRESSING RELATIONS BETWEEN $\frac{z'}{c}$ and $\frac{1}{U'}$ or $\frac{1}{V'}$.

Substituting the value found for U in Equation (30) in Equations (29), and re-arranging,

$$\left.\begin{aligned}
0 &= k\left[\sum \frac{M_x^{\,m}}{\left(\frac{z'}{c}\,U'\right)^m}\,\frac{y\,\Delta x}{b^m\,c^{1+2m}\cos\alpha} + \sum \frac{u'}{c}\,\frac{M_x^{\,m}}{\left(\frac{z'}{c}\,U'\right)^m}\,\frac{\Delta x}{b^m\,c^{2m}}\right] \\
0 &= k\left[\sum \frac{M_x^{\,m}}{\left(\frac{z'}{c}\,U'\right)^m}\,\frac{x\,\Delta x}{b^m\,c^{1+2m}\cos\alpha} + \sum \frac{u'}{c}\,\frac{M_x^{\,m}}{\left(\frac{z'}{c}\,U'\right)^m}\,\frac{\tan\alpha\,\Delta x}{b^m\,c^{2m}}\right] \\
0 &= k\sum \frac{M_x^{\,m}}{\left(\frac{z'}{c}\,U'\right)^m}\,\frac{\Delta x}{b^m\,c^{1+2m}\cos\alpha}
\end{aligned}\right\} \dots(32)$$

The factor, k, may be dropped, of course, from all these equations unless some element, such as shrinkage or temperature change, is taken into account.

Analyses of arches under various particular conditions of loading and deformation have been carried out by the proposed theory of elasticity, using Equations (32) and Figs. 9 and 10.

Importance of Variable Modulus of Elasticity.—The complexity of the angle changes in an arch is probably best realized by a few preliminary investigations. Upon first thought, one might conclude that the secant modulus for the average working stress can not be far wrong. However, the combined action of compression and bending moment introduces complications which are probably not immediately anticipated.

As an example, consider Fig. 11. The curve indicates the differential angle change at a given section due to a constant bending moment, the only variation being the eccentricity of the force producing the moment. It has been plotted on the basis of the exponential equation for the relation of stress to strain, considering $m = 1.3$. The ordinates are plotted as a constant, f, times $\frac{d\,\phi}{d\,s}$. The values of points on the curve are readily found by the aid of Fig. 10. If a constant secant modulus were adopted, and Hooke's law assumed to apply, the curve would take the form of a vertical straight line.

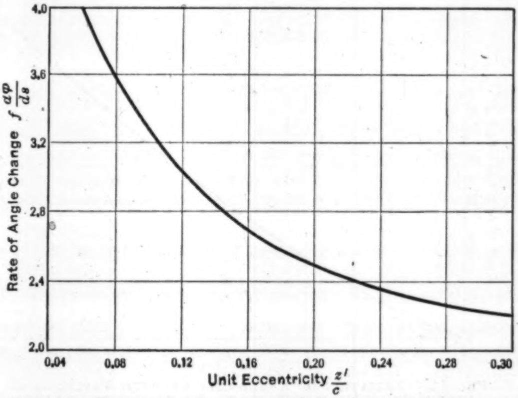


FIG. 11.—VARIATION OF ANGLE CHANGE WITH CHANGE IN ECCENTRICITY OF FORCE. MOMENT CONSIDERED CONSTANT. $m = 1.3$.

These considerations would indicate that no satisfactory study of the effect of a variable modulus of elasticity on the action of arches can be made, unless the eccentricity of the line of thrust as well as the bending moment is included in the analysis.

Consider a simple example solved by means of Fig. 11. No particular units will be kept in mind. Assume a load of 20 applied on a specimen, at an eccentricity, $0.06\,\frac{z'}{c}$. When $m = 1.3$, the rotation will be,

$$\phi = (20 \times 0.06)^{1.3} \times 3.95 = 5.00$$

Assume, now, that the same rotation is obtained, in a specimen of the same shape, but having $m = 1$ as an index of the stress-strain relation, for a load of 20 placed at the same eccentricity.

Now, consider the loads removed, and a load of 3 placed at an eccentricity, $0.26 \frac{z'}{c}$, on both specimens. The rotation will be:

For $m = 1$,

$$\phi = 5.00 \times \frac{3 \times 0.26}{20 \times 0.06} = 3.25$$

and for $m = 1.3$,

$$\phi = (3 \times 0.26)^{1.3} \times 2.3 = 1.66$$

Hence, while the rotation in both specimens may be the same for one condition of loading, it may be quite different for another condition even though there is little change in bending moment. This analysis corresponds to taking a secant modulus for one case, that is, when $m = 1$, and the exponential formula in the other case.

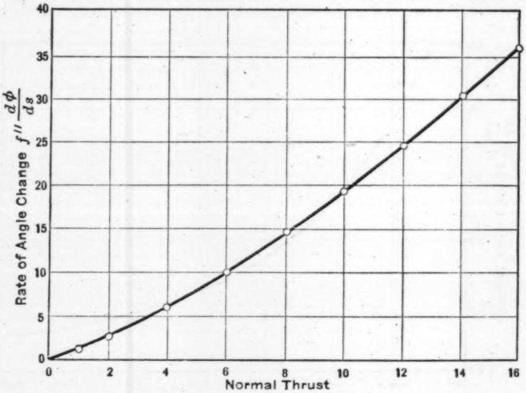


FIG. 12.—VARIATION OF ANGLE CHANGE WITH VARIATION OF INTENSITY OF NORMAL THRUST. ECCENTRICITY OF LINE OF THRUST CONSIDERED CONSTANT, $m = 1.3$.

A further investigation has been plotted in Fig. 12. The eccentricity of the line of thrust through a particular section was assumed to remain the same, while the normal thrust was considered to vary. The value of m in the exponential formula was assumed to be equal to 1.3. The rate of angle change is plotted in terms of $\frac{d\,\phi}{d\,s}$ times a constant, f''.

Again, Fig. 13 shows the results of a study of the variation of angle change with the eccentricity of the line of thrust, considering the normal thrust to remain constant. The full line is drawn through points computed with the aid of Fig. 10; the broken line is a straight line drawn through the origin of co-ordinates at a slope arbitrarily chosen to fit closely the computed curve. The rate of angle change is plotted as $\frac{d\,\phi}{d\,s}$ times a constant, f'.

A comparison of Figs. 11, 12, and 13 shows how the action of a structure may be anticipated. Fig. 12 indicates that the rate of angle change at a section increases more rapidly than the thrust, while Fig. 11 shows that, for a given moment, the rate of angle change decreases with an increase in eccentricity of the thrust producing the moment.

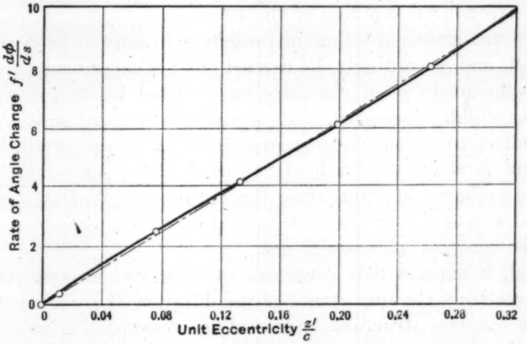


FIG. 13.—VARIATION OF ANGLE CHANGE WITH ECCENTRICITY OF LINE OF THRUST. NORMAL THRUST CONSIDERED CONSTANT. $m = 1.3$.

If the thrust remains constant (Fig. 13), the angle change varies almost directly as the eccentricity; that is, the curve representing the relation between the rate of angle change and the eccentricity is very nearly a straight line. Hence, considering an arch so proportioned that for a given loading the average stress, $\frac{N}{A}$, is the same on every normal section, and, neglecting rib-shortening effects, it might be expected that the ordinary theory of elasticity would give the location of the line of thrust very closely. However, if the average normal stress varies considerably from section to section, the line of thrust determined by the ordinary theory of elasticity could not be expected to be quite as accurate.

As a matter of fact the average compressive stress on the various normal sections throughout an arch of usual design does not vary to a large degree. For a uniformly loaded symmetrical parabolic arch, in which the sectional area increases as the secant of the angle between a tangent to the rib axis and the horizontal, the average compressive stress would be the same for every normal section.

III.—PROPOSED THEORY OF PLASTICITY

That property of concrete by which the material undergoes a gradual but persistent deformation while in the condition of sustained stress, has been the subject of considerable discussion during the last decade. This phenomenon of plasticity, often referred to as "flow of concrete" or "time-yield", has been recognized by the early concrete experimenters, but only comparatively recently has it been given special attention in investigations.

In the proposed theory of elasticity the total strain is an exponential function of the stress and independent of the time interval of the stress. In contradistinction that group of mathematical relations which treats of the deformations in a structure where the total strain is a function of the duration as well as intensity of stress, has been arbitrarily termed the "theory of plasticity". As in the proposed theory of elasticity, this group of relations accounts for strain as an exponential function of the stress.

C. Bach[11] was among the first investigators to notice the phenomenon of permanent set, and, accordingly, he carried out his experiments for the determination of the modulus of elasticity by repeated loading, each successive loading being slightly increased. His modulus was based upon the rebound.

As a result of numerous German experiments,[12] it was concluded that the permanent set is about one-tenth of the combined deformation. The latter conclusion is probably in error, since the duration of loading was not great. Experiments reported by G. Magnel[13] indicate that the permanent set becomes smaller as the concrete increases in age.

In general, it appears that American investigators in concrete have been foremost in realizing the importance of the duration of stress upon the deformations in a concrete structure. Some of the American experiments will be discussed in detail.

Proposed Exponential Equation of Plastic Deformation.—Investigations in the plasticity of concrete, for the most part, have been of a qualitative, rather than quantitative, nature. The ultimate test of all experimental data on a material of engineering construction is the adaptability of such data to transformation into true mathematical laws which will define the properties of the material in such a manner that they may be accounted for in the analysis of complete structures.

Study of test data indicates that, while a mathematical law defining the plasticity of concrete must be expressed in three variables, it is closely given by an exponential equation involving the three variables, two of which are independent.

Investigations of a qualitative nature have been published by F. R. McMillan, A. H. Fuller, and H. F. Clemmer, Members, Am. Soc. C. E., and S. C. Hollister, Assoc. M. Am. Soc. C. E., as well as by Messrs. M. B. Lagaard, E. B. Smith, C. O. More, and others. The data made available by these experiments have been used by the writer as a comparison to the exponential equation,

$$\rho = K\,\sigma^p\,t^q$$

in which, ρ is the plastic deformation which is a function of the unit stress and the time, t. The factor, K, is a parameter depending on the properties of the material. The exponents, p and q, are constants which depend on the properties of the material at the beginning of the stressed condition.

---

[11] *Zeitschrift* des Vereines deutscher Ingenieure, 1895, p. 489.

[12] *Deutscher Ausschuss für Eisenbeton*, Heft 17 (1912), pp. 66–67.

[13] "Recherches experimentales sur le coefficient d'élasticité du béton," G. Magnel, *Revue Universelle des Mines*, Vol. 67, No. 1, 1924.

In proposing this equation, it is realized that the elastic and plastic properties of concrete vary considerably with the age of the material, especially during the first two or three months after it has been mixed. The values, K, p, and q, are intended to define the law of time-stress yield of the concrete during any time interval, provided the parameters satisfy the properties of the material at the beginning of the interval. The variable, t, is measured with its zero value at the instant that the load producing a stress, σ, is applied.

It will be observed that p and q are independent of the units of measurement of the stress, time, or deformation. Only K will be affected by a change in the units of measure.

Experimental Basis for Proposed Exponential Equation of Plastic Deformation.—Assuming that the plasticity of concrete may be expressed in terms of an exponential equation in which the time and stress intensity are independent variables, experimental data have been plotted on logarithmic paper to find the variable relation between deformation and time, for a constant stress. Similarly, experimental values have been plotted to determine the relation between deformation and stress for constant time intervals.

Many of the test data available have not been compiled in such a way that the effects of shrinkage and temperature deformations can be separated from the plastic deformations. However, these effects in the data considered are believed to be small enough not to affect the conclusions. No attempt has been made to convert all data examined to the same units, because only K is affected.

Time tests by Fuller and More[14] were performed on specimens presumably of approximately the same quality of concrete. The data have been plotted in Fig. 14. Specimens A and B were cylinders 8 in. in diameter, made from concrete mixed for use in building construction. Specimen C was a block 8 in. square and 13 in. high, cut from a floor of the same building. All specimens were subjected to different loadings before applying a constant load over a long period of time. Tests on Cylinder A were begun 101 days after pouring, and the time-yield test under a stress of 650 lb. per sq. in. was begun 24 days later. Cylinder B was 137 days old when tests were begun; a constant load causing a stress of 1 150 lb. per sq. in. was applied after 6 days of variable load tests. Specimen C was 131 days old, and the constant-load time-yield test was begun after 11 days, with a stress of 650 lb. per sq. in. No corrections have been made for temperature and moisture variations or for deformation due to shrinkage. However, comparatively little shrinkage will occur after 4 months, which time was the approximate age of the specimen. The tests having been made under laboratory conditions, it is believed that temperature and moisture variations were too small to affect the results materially.

No data were reported from which a relation between the intensity of stress and rate of deformation could be computed.

The exponential relation for time-yield for these experiments becomes,

$$\rho = f\ (\sigma^p)\ t^{0.42}$$

---

[14] "Time Tests of Concrete," by Fuller and More, *Proceedings*, Am. Concrete Inst., Vol. 12, p. 302, 1916.

In spite of the fact that all specimens were treated differently in the early stages of the load tests and one was loaded with a greater stress intensity than the others, all deformed approximately as the 0.42 power of the time. The theoretical curves shown in Fig. 14 fit the experimental points fairly closely.

Mr. Earl B. Smith made several tests[15] to determine the rate of yield of stressed concrete, part of which were on a reinforced concrete beam of 1:2:4 mix with gravel as the coarse aggregate. Reinforcement consisted of five ½-in. rods. The beam was 5 in. wide, 8 in. deep, including 1-in. steel cover, and had a clear span of 10 ft. About two weeks after pouring, the beam was loaded at the center with a concentrated load of 1 225 lb. Deformations were measured at the top of the beam by means of a strain-gauge. Since, for loads considerably below the ultimate, the outer fiber stress in a concrete section of a beam is approximately proportional to the moment, the relation of deformation and stress was found approximately by comparing the moment with the deformation.

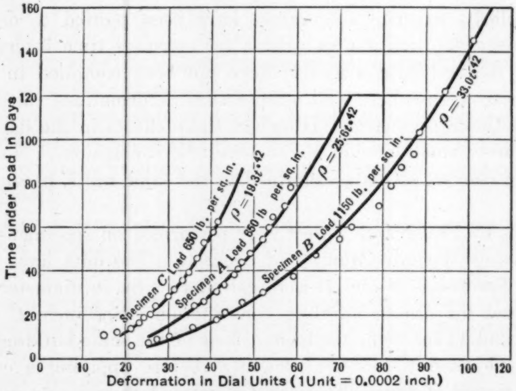


FIG. 14.—TIME-YIELD TESTS BY FULLER AND MORE. CURVES BY PROPOSED EXPONENTIAL FORMULA.

It is realized, of course, that such measurements would be true only if the plastic deformation varies directly as the stress. Since the plastic deformation appears to vary more nearly as the 1.3 power of the stress, there is likely to be a redistribution of stress over the section as the period of loading continues, the stress decreasing near the outer fiber and increasing near the neutral axis. The neutral axis will move down slightly, because the concrete continues to deform while the steel remains unchanged for a given stress. Hence, some error is introduced by assuming the outer fiber stress constant during a time-yield test. However, this should not seriously affect the general law which is being derived subject to confirmation by more precise experiments.

As the beam was only two weeks old when the test was started, considerable deformation was due to shrinkage. Correction was made for this by subtracting the deformation over the support from the recorded plastic flow throughout the span, for each time interval.

---

[15] "Flow of Concrete Under Sustained Loads," by E. B. Smith, *Proceedings*, Am. Concrete Inst., Vol. 13, p. 99, 1917; Vol. 12, p. 317, 1916.

The exponential equation, $\rho = K\ \sigma^p\ t^q$, for the concrete in Mr. Smith's test beam, takes the form,

$$\rho = 0.198\ \sigma^{1.73}\ t^{0.18}$$

in which, ρ is the deformation in 0.0001 in., measured over an 8-in. length; σ is the fiber stress measured in terms of the moment, in thousands of foot-pounds; and t is the time of the period of stress, in days. A change to any other units of measure will simply change the coefficient of the second member of the equation.

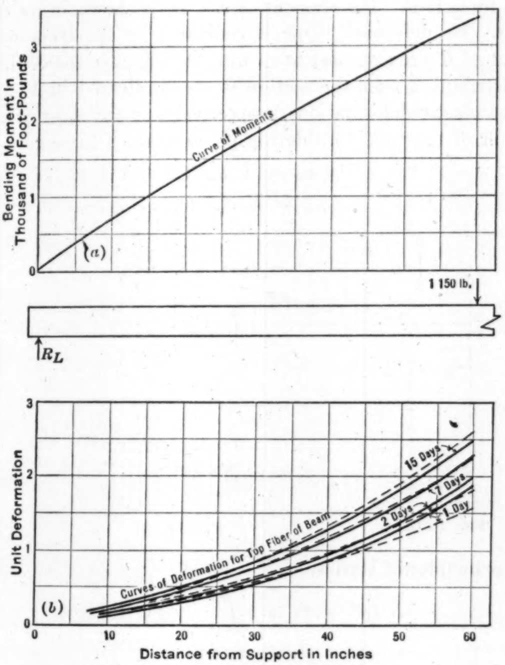


FIG. 15.—EXPERIMENTS BY E. B. SMITH.

The moment curve (Fig. 15(a)) is for combined dead and live loads. The graphs (Fig. 15(b)) have been plotted from experimental curves, indicated by Mr. Smith, by making correction for the shrinkage. Superimposed are a series of curves, in broken lines, indicating the plastic flow determined by the proposed exponential formula.

At the center of the beam, it will be noticed, the theoretical curves do not agree very closely with the experimental curves, especially for the short time intervals. It seems possible that this may be due in part to the localized effect of the concentrated load.

DERIVATION OF EXACT EQUATIONS FOR THE PLASTIC THEORY

General Requirements.—As with the analysis for a variable modulus of elasticity, so for the proposed theory of plasticity—it is not possible to superimpose the effects of individual forces separately upon a structure to obtain the effect of the combined system. In addition, the theory of plasticity presents further difficulty. Except in special cases, the elastic deformations and plastic deformations are dependent on each other; and, to be perfectly exact, the total deformation may not be written as the sum of two explicit functions of the elastic and plastic deformation, respectively.

Derivation of Equations.—Consider a column (Fig. 16(*a*)), measuring *b* by *d* in section, to be loaded by an eccentric normal force, *N*. The eccentricity of the force is *z′*. The element, *A B C D,* is shown in an enlarged scale in Fig. 16(*b*). Its deformed shape is indicated by the trapezoid, *A′ B′ C D.* Lines *A B* and *A′ B′* are extended to an intersection at 0, the center of rotation. The stress distribution over the section is also indicated in Fig. 16(*b*). The elastic and the plastic deformations, respectively, of any fiber at a distance, *w*, from the center of rotation, considering plane sections to remain plane, are,

$$\epsilon_w = k\ \sigma_w^{m} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

$$\rho_w = K\ \sigma_w^{p}\ t^{q} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

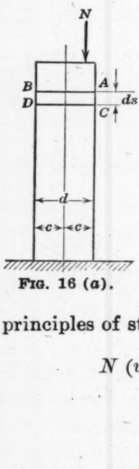


FIG. 16 (*a*).

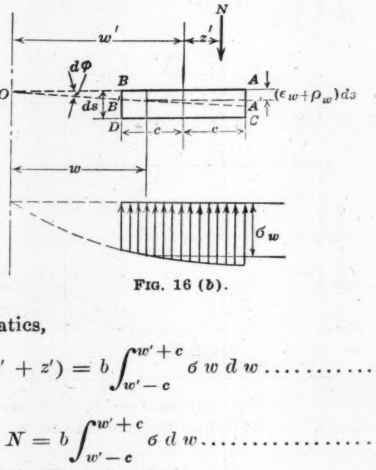


FIG. 16 (*b*).

From the principles of statics,

$$N\ (w' + z') = b \int_{w'-c}^{w'+c} \sigma\ w\ d\ w \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

and,

$$N = b \int_{w'-c}^{w'+c} \sigma\ d\ w \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

Thus, dividing Equation (35) by Equation (36),

$$(w' + z') = \int_{w'-c}^{w'+c} \sigma\ w\ d\ w \div \int_{w'-c}^{w'+c} \sigma\ d\ w \ldots\ldots\ldots (37)$$

From the geometry of Fig. 16(*b*) and Equations (33) and (34),

$$\left.\begin{aligned} w\,\frac{d\ \phi}{d\ s} &= \varepsilon_w + \rho_w = k\ \sigma_w^{m} + K\ \sigma_w^{p}\ t^{q} \\ w &= \frac{1}{\dfrac{d\ \phi}{d\ s}}\left[k\,\sigma_w^{m} + K\,\sigma_w^{p}\ t^{q}\right] \\ d\ w &= \frac{1}{\dfrac{d\ \phi}{d\ s}}\ \left[m\ k\ \sigma_w^{m-1} + p\ K\ \sigma_w^{p-1}\ t^{q}\right] \end{aligned}\right\} \ldots\ldots\ldots\ldots (38)$$

Considering $\frac{d\ \phi}{d\ s}$ a parameter, values of w and $d\ w$ from Equations (38) substituted into Equations (35) and (36) result in,

$$N\,(w' + z') = \frac{1}{\left(\frac{d\ \phi}{d\ s}\right)^2} \int_{\sigma_2}^{\sigma_1} \sigma\,[k\ \sigma^m + K\ \sigma^p\ t^q]\ [m\ k\ \sigma^{m-1} + p\ K\ \sigma^{p-1}\ t^q]\ d\,\sigma$$

$$= \frac{1}{\left(\frac{d\ \phi}{d\ s}\right)^2} \left[\int_{\sigma_2}^{\sigma_1} m\,k^2 \sigma^{2m}\,d\,\sigma + \int_{\sigma_2}^{\sigma_1} (m + p)\,k\,K\,t^q\,\sigma^{p+m}\,d\,\sigma + \int_{\sigma_2}^{\sigma_1} p\,K^2\,\sigma^{2p}\,t^{2q}\,d\,\sigma\right]$$

which when integrated becomes,

$$N\,(w' + z') = \frac{1}{\left(\frac{d\ \phi}{d\ s}\right)^2} \left[\frac{m\ k^2}{2\ m + 1}\,({\sigma_1}^{2\,m+1} - {\sigma_2}^{2\,m+1}) + \frac{(m + p)\,k\,K\,t^q}{(p + m + 1)}\,({\sigma_1}^{(p+m+1)} - {\sigma_2}^{(p+m+1)}) + \frac{p\ K^2\ t^{2q}}{2\ p + 1}\,({\sigma_1}^{2\,p+1} - {\sigma_2}^{2\,p+1})\right]$$

or,

$$N\ (w' + z') = \frac{F}{\left(\frac{d\ \phi}{d\ s}\right)^2} \ldots\ldots\ldots\ldots (39)$$

in which,

$$F = \left[\frac{m\ k^2}{2\ m + 1}\,({\sigma_1}^{2\,m+1} - {\sigma_2}^{2\,m+1}) + \frac{(m + p)\ k\ K\ t^q}{p + m + 1}\,({\sigma_1}^{(p+m+1)} - {\sigma_2}^{(p+m+1)}) + \frac{p\ K^2\ t^{2\,q}}{2\ p + 1}\,({\sigma_1}^{2\,p+1} - {\sigma_2}^{2\,p+1})\right] \ldots\ldots\ldots\ldots (40)$$

By a similar process it will be found that,

$$N = \frac{G}{\frac{d\ \phi}{d\ s}} \ldots\ldots\ldots\ldots (41)$$

in which,

$$G = \frac{m\ k}{m + 1}\,({\sigma_1}^{m+1} - {\sigma_2}^{m+1}) + \frac{p\ K\ t^q}{p + 1}\,({\sigma_1}^{p+1} - {\sigma_2}^{p+1}) \ldots\ldots\ldots\ldots (42)$$

From Equations (39) and (41) may be obtained the relation,

$$\frac{d\ \phi}{d\ s} = \frac{F}{G\ (w' + z')} \ldots\ldots\ldots\ldots (43)$$

Special Case, $t = 0$.—Immediately after the load is applied, that is, the special case when $t = 0$, w' becomes equal to u' and Equation (43) reduces to,

$$\frac{d\ \phi_E}{d\ s} = k\,\frac{m + 1}{2\ m + 1}\,\frac{({\sigma_1}^{2\,m+1} - {\sigma_2}^{2\,m+1})}{({\sigma_1}^{m+1} - {\sigma_2}^{m+1})} \times \frac{1}{u' + z'} \ldots\ldots\ldots\ldots (44)$$

but,

$$\sigma = \left(\frac{\varepsilon}{k}\right)^{\frac{1}{m}} \ldots\ldots\ldots\ldots (45)$$

also,

$$\varepsilon_1 = (u' + c)\,\frac{d\ \phi_E}{d\ s} \text{ and } \varepsilon_2 = (u' - c)\,\frac{d\ \phi_E}{d\ s} \ldots\ldots\ldots\ldots (46)$$

hence, from Equations (45) and (46),

$$\sigma_1 = \frac{\left[(u' + c)\,\dfrac{d\,\phi_E}{d\,s}\right]^{\frac{1}{m}}}{k^{\frac{1}{m}}} \text{ and } \sigma_2 = \frac{\left[(u' - c)\,\dfrac{d\,\phi_E}{d\,s}\right]^{\frac{1}{m}}}{k^{\frac{1}{m}}} \ldots\ldots\ldots (47)$$

Substituting the values of σ_1 and σ_2 from Equations (47) into Equation (44) and re-arranging terms,

$$(u' + z') = \frac{m + 1}{2\,m + 1}\;\frac{\left[(u' + c)^{\frac{1+2\,m}{m}} - (u' - c)^{\frac{1+2\,m}{m}}\right]}{\left[(u' + c)^{\frac{1+m}{m}} - (u' - c)^{\frac{1+m}{m}}\right]} \ldots\ldots (48)$$

which is identical with Equation (22) for the proposed theory of elasticity with E introduced as a variable.

Development of the Theory of Plasticity for the Arch

General Assumptions.—The total rate of angle change, $\frac{d\,\phi}{d\,s}$, given by Equation (43), is very complicated and would prove extremely unwieldy, if not impossible, to apply to a structure such as the concrete arch. The unknowns are in terms of the extreme fiber stresses which, according to the theory, will vary with time even should the load and eccentricity of the load remain constant.

To obviate this difficulty the following equation is introduced,

$$\frac{d\,\phi}{d\,s} = \frac{d\,\phi_E}{d\,s} + \frac{d\,\phi_P}{d\,s} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (49)$$

as a fundamental basis for further analysis. In this relation, $\frac{d\,\phi}{d\,s}$ is the total rate of rotation at a given section; $\frac{d\,\phi_E}{d\,s}$ is the rate of elastic rotation at the section; while $\frac{d\,\phi_P}{d\,s}$ is the rate of plastic rotation at the end of a particular time interval.

The theory, as here presented, is not quite exact, for it is developed on the assumption that the center of plastic rotation for each section is independent of the elastic properties of the concrete. To be perfectly exact, the center of plastic rotation at a section at the beginning of the stressed condition coincides with the center of elastic rotation, and moves toward the ultimate center of plastic rotation as the time interval of stress continues and redistribution of stresses within the section takes place. The theory is exact when the rate of change of elastic deformation with variation in stress is equal to the rate of change of plastic deformation (that is, when the parameters, m and p, are equal), because in this case no redistribution of stresses takes place, and the center of elastic rotation coincides with the center of plastic rotation.

However, since the location of the ultimate center of plastic deformation approximates that of the center of elastic deformation for the probable range of values of m and p, no large error is introduced by assuming that there is an instantaneous movement of the center of rotation from the elastic center to the ultimate plastic center.

The group of relations derived from Equation (49) will satisfy all the laws of statics and conditions of continuity; hence, they are mathematically correct. The term, $\frac{d\,\phi_E}{d\,s}$, of the second member of Equation (49) will be the same as for the proposed theory of elasticity.

Although the derivation which follows is for combined compression and flexure, Equation (49) may be expanded equally well for pure flexure.

Derivation of Simplified Equations of Plasticity.—Consider (Fig. 17(*a*)) an elementary radial segment, $d\,s$, of an arch. The center of the element is located at a point the abscissa of which is x. It is subjected to a normal thrust, N_x, acting at a distance, z', from the centroidal axis.

An enlarged diagram of the element is indicated in Fig. 17(*b*). The elastic deformation of the segment is indicated by the trapezoid, $A\,B\,B'\,A'$; the plastic deformation is $A'\,B'\,B''\,A''$. Point O is the center of elastic rotation, while Point O' is considered to be the center of plastic rotation. Other nomenclature is indicated in the diagram.

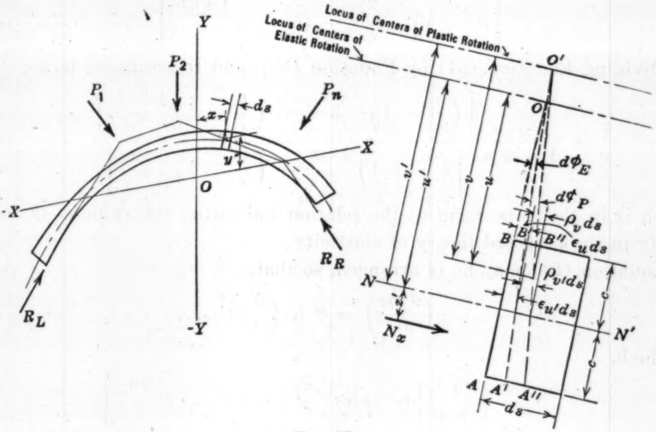


FIG. 17.

To determine $\frac{d\,\phi_P}{d\,s}$, the plastic deformation of a fiber at a distance, v, from the center of plastic rotation, O', assuming stress distribution over the section to be dependent on the plastic properties of the material, is,

$$\rho_v = K\,\sigma_v^{\,p}\,t^q \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (50)$$

Geometric relations show that,

$$\rho_v = v\,\frac{d\,\phi_P}{d\,s} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

Eliminating ρ_v between Equations (50) and (51), and re-arranging terms,

$$\sigma_v = \frac{1}{(t)^{\frac{q}{p}}}\left(\frac{v}{K}\,\frac{d\,\phi_P}{d\,s}\right)^{\frac{1}{p}} \quad\ldots\ldots\ldots\ldots\ldots\ldots (52)$$

Since, for static equilibrium, the sum of the forces acting on the section in line with $n\,n'$ must be zero,

$$N_x = \int_{(v'-c)}^{(v'+c)} \sigma_v \, d\,A = \frac{b}{(t)^{\frac{q}{p}}\, K^{\frac{1}{p}}} \left(\frac{d\,\phi_P}{d\,s}\right)^{\frac{1}{p}} \int_{(v'-c)}^{(v'+c)} v^{\frac{1}{p}}\, d\,v$$

from which,

$$N_x = \frac{p}{1+p}\,\frac{b}{K^{\frac{1}{p}}\, t^{\frac{q}{p}}} \left(\frac{d\,\phi_P}{d\,s}\right)^{\frac{1}{p}} \left[(v'+c)^{\frac{1+p}{p}} - (v-c)^{\frac{1+p}{p}}\right] \ldots (53)$$

For equilibrium against static rotation,

$$N_x\,(v'+z') = \frac{b}{K^{\frac{1}{p}}\, t^{\frac{q}{p}}} \left(\frac{d\,\phi_P}{d\,s}\right)^{\frac{1}{p}} \int_{(v'-c)}^{(v'+c)} v^{\frac{1+p}{p}}\, d\,v$$

which, when integrated, becomes,

$$N_x\,(v'+z') = \frac{p}{1+2\,p}\,\frac{b}{K^{\frac{1}{p}}\, t^{\frac{q}{p}}} \left(\frac{d\,\phi_P}{d\,s}\right)^{\frac{1}{p}}$$

$$\left[(v'+c)^{\frac{1+2\,p}{p}} - (v'-c)^{\frac{1+2\,p}{p}}\right] \ldots\ldots\ldots\ldots\ldots\ldots (54)$$

Dividing Equation (54) by Equation (53), and re-arranging terms,

$$\frac{z'}{c} = \frac{1+p}{1+2\,p}\;\frac{\left[\left(\frac{v'}{c}+1\right)^{\frac{1+2\,p}{p}} - \left(\frac{v'}{c}-1\right)^{\frac{1+2\,p}{p}}\right]}{\left[\left(\frac{v'}{c}+1\right)^{\frac{1+p}{p}} - \left(\frac{v'}{c}-1\right)^{\frac{1+p}{p}}\right]} - \frac{v'}{c} \ldots (55)$$

which is in the same form as the relation indicating the value of the eccentricity in the proposed theory of elasticity.

Equation (53) may be re-arranged, so that,

$$\left(\frac{d\,\phi_P}{d\,s}\right) = t^q\,K\left(\frac{N_x}{V}\right)^p \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (56)$$

in which,

$$V = \frac{b\,p}{1+p}\left[(v'+c)^{\frac{1+p}{p}} - (v'-c)^{\frac{1+p}{p}}\right] \ldots\ldots\ldots\ldots (57)$$

Equations (56) and (57) are of the same form as Equations (24) and (25) in the proposed theory of elasticity. Equation (56) may also be written in the form,

$$\left(\frac{d\,\phi_P}{d\,s}\right) = t^q\,K\,\frac{M^p}{(z'\,V)^p} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (58)$$

The rib-shortening of the elemental length, $d\,s$, due to plastic flow, is,

$$d\,\xi_P = \rho_{v'}\,d\,s$$

Geometrically,

$$\rho_{v'} = v'\,\frac{d\,\phi_P}{d\,s}$$

hence,

$$\frac{d\,\xi_P}{d\,s} = v'\,\frac{d\,\phi_P}{d\,s}$$

or,

$$\frac{d\,\xi_P}{d\,s} = v'\,t^q\,K\left(\frac{N_x}{V}\right)^p \text{..........................}(59)$$

The total rate of rotation at a section of the arch is now given by the expression,

$$\frac{d\,\phi}{d\,s} = \frac{d\,\phi_E}{d\,s} + \frac{d\,\phi_P}{d\,s}$$

or,

$$\frac{d\,\phi}{d\,s} = k\left(\frac{N_x}{U}\right)^m + t^q\,K\left(\frac{N_x}{V}\right)^p \text{.....................}(60)$$

Also, the total rate of rib-shortening, is,

$$\frac{d\,\xi}{d\,s} = \frac{d\,\xi_E}{d\,s} + \frac{d\,\xi_P}{d\,s}$$

or,

$$\frac{d\,\xi}{d\,s} = u'\,k\left(\frac{N_x}{U}\right)^m + v'\,t^q\,K\left(\frac{N_x}{V}\right)^p \text{..................}(61)$$

Consider, now, a symmetrical arch, and neglect shear deformations. The dead load on the arch will be symmetrical; hence, rib-shortening can cause no tendency for relative vertical displacement of the abutments. Substituting Equations (60) and (61) into Equations (5), which indicate the geometrical relations of the change in shape of the arch, the following equations are obtained:

$$\left.\begin{aligned} 0 &= k\int\left(\frac{N}{U}\right)^m \frac{y\,d\,x}{\cos\alpha} + t^q\,K\int\left(\frac{N}{V}\right)^p \frac{y\,d\,x}{\cos\alpha} \\ &\quad + k\int u'\left(\frac{N}{U}\right)^m d\,x + t^q\,K\int v'\left(\frac{N}{V}\right)^p d\,x \\ 0 &= k\int\left(\frac{N}{U}\right)^m \frac{x\,d\,x}{\cos\alpha} + t^q\,K\int\left(\frac{N}{V}\right)^p \frac{x\,d\,x}{\cos\alpha} \\ 0 &= k\int\left(\frac{N}{U}\right)^m \frac{d\,x}{\cos\alpha} + t^q\,K\int\left(\frac{N}{V}\right)^p \frac{d\,x}{\cos\alpha} \end{aligned}\right\}\text{..........}(62)$$

While it is entirely possible to combine these equations with the static relations, the result would be exceedingly complicated. Since Maxwell's law of reciprocal deflections has no meaning in these equations, no simplification can be introduced by special arrangement of the co-ordinate system, or the like.

Equations (62) are derived on the basis of a statically determinate structure, that is, the location of the line of thrust in the arch must be known before the angle changes can be computed. For example,

$$t^q\,K\int\left(\frac{N}{V}\right)^p \frac{y\,d\,x}{\cos\alpha}$$

is the relative movement of the ends of the imaginary cantilevers terminating at the center of co-ordinates, along the x-axis.

Equations (62) may be solved with the aid of Figs. 9 and 10 in a manner similar to that explained for the proposed theory of elasticity.

Effect of Plastic Flow in an Arch When $m = p$.—If m and p are equal to the same number, n, that is, $n = m = p$, it will be found there are common values, w and w', such that $W = U = V$ and $w' = u' = v'$.

Equations (62) may then be written in the form,

$$\left.\begin{aligned} 0 &= (k + t^q\, K) \left\{ \int_R^L \left(\frac{N}{W}\right)^n \frac{y\, d\, x}{\cos\alpha} + \int_R^L \left(\frac{N}{W}\right)^n w'\, d\, x \right\} \\ 0 &= (k + t^q\, K) \int_R^L \left(\frac{N}{W}\right)^n \frac{x\, d\, x}{\cos\alpha} \\ 0 &= (k + t^q\, K) \int_R^L \left(\frac{N}{W}\right)^n \frac{d\, x}{\cos\alpha} \end{aligned}\right\} \ldots\ldots(63)$$

It will be noted from Equations (63) that, in case the exponents, m and p, are equal, there can be no relative movement of the line of thrust due to plastic flow. Thus, although an arch undergoes a comparatively large amount of rib-shortening within a time interval, because of flow of the concrete, this is no reason for concluding that there has been a large redistribution of moments or stresses.

An actual structure, of course, presents greater complications than those indicated, for shrinkage, temperature changes, and live loads of varying intensity act in conjunction with the constant dead load pressures.

THE THEORY OF PLASTICITY FOR PURE FLEXURE

General Equation.—The rate of angle change at a section of a member subjected to both direct and flexural stresses is dependent on the intensity of the direct thrust as well as on the bending moment. Hence, Equation (49) takes a special form when developed for flexure alone.

A derivation of the flexure formulas for plasticity is similar to the development presented in the proposed theory of elasticity for flexure. It will be found that,

$$\frac{d\, \phi_P}{d\, s} = K \left(\frac{M}{S}\right)^p t^q \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(64)$$

in which,

$$S = \frac{2\, p}{1 + 2\, p}\, b\, c^{\frac{1 + 2\, p}{p}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(65)$$

Other terms in Equation (64) have been defined in the development of the theory of plasticity for arches.

Substituting the values of $\frac{d\, \phi_E}{d\, s}$ and $\frac{d\, \phi_P}{d\, s}$, found, respectively, in Equations (13) and (64), into Equation (49), the relation for the total rate of angle change becomes,

$$\frac{d\, \phi}{d\, s} = k \left(\frac{M}{Q}\right)^m + K \left(\frac{M}{S}\right)^p t^q \ldots\ldots\ldots\ldots\ldots\ldots(66)$$

As in the case of the theory of plasticity for arches, this relation is physically exact only when $n = m = p$. In that case, $Z = Q = S$, and Equation (66) may be rewritten as,

$$\frac{d\, \phi}{d\, s} = (k + t^q\, K) \left(\frac{M}{Z}\right)^n \ldots\ldots\ldots\ldots\ldots\ldots(67)$$

IV.—ANALYSIS OF STRUCTURES IN PURE FLEXURE

Concrete structures designed for flexure can be economical only when combined with steel, for concrete has a very low tensile strength. However, if fiber stresses were kept sufficiently low, it would be entirely possible to design concrete beams to support loads by pure flexure.

What follows covers studies of beams composed of a material having elastic and plastic properties similar to that indicated by the experimental data on concrete, and shows their application to structures in pure flexure and the effects to be expected because of such phenomena as plastic flow. In addition, computations have been made indicating what effect a variable relation of stress to strain has on the distribution of stress and moments in a loaded structure.

No particular attempt has been made to choose exact constants to define the properties of the material in the exponential equations, but values have been chosen which will probably give the extreme conditions found in practice. Before very close values can be assigned to the parameters in the exponential equations, more precise experiments must be performed. This is especially true for the proposed plastic equations.

Deflection of Simple Supported Beam with Concentrated Load at Center.—Fig. 18 indicates a simple supported beam, of length, l, subjected to a concentrated load, P, at mid-span. The beam will be assumed to be 10 in. wide and 40 in. deep. Consider that both elastic and plastic deformations take place under the influence of the load, and that the parameters in the exponential equations have the following values: $m = p = n = 1.15$; $k = 183 \times 10^{-9}$; $K = 17 \times 10^{-9}$; and $q = 0.40$.

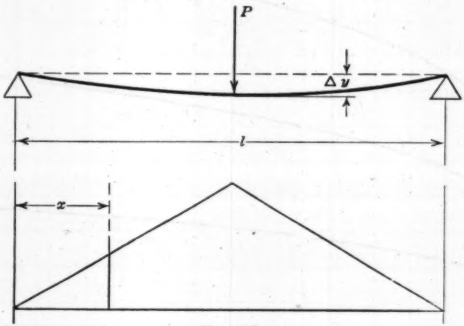


FIG. 18.

Since the beam is symmetrical, the tangent to the beam under the load will remain parallel to a line through the supports. Hence, the center deflection may be expressed by,

$$\Delta y = \int_0^{\frac{1}{2}l} \frac{d\,\phi}{d\,s}\,x\,d\,x$$

but,

$$\frac{d\,\phi}{d\,s} = (k + t^q\,K)\left(\frac{M}{Z}\right)^n$$

The bending moment at any point along the beam is,

$$M = \frac{1}{2}\,P\,x$$

hence,

$$\Delta\,y = (k + t^q\,K)\left(\frac{\frac{1}{2}\,P}{Z}\right)^n \int_0^{\frac{1}{2}l} x^{(1+n)}\,d\,x$$

Integrating and substituting numerical values, where given,

$$\Delta\,y = P^{1.15}\,\frac{(183 + 17\,t^{0.4})\,10^{-13}}{1\,135}\,l^{3.15}$$

In order that numerical results may be obtained, the length, *l*, of the beam will be considered to be 250 in. The last equation may then be rewritten as,

$$\Delta\,y = P^{1.15}\,(57.2 + 5.32\,t^{0.4})\,10^{-8}$$

The deflections at the ends of various time intervals and for different magnitudes of the concentrated load, are plotted in Fig. 19. The ordinates corresponding to the respective load concentrations on the line of zero abscissas measure the deflection at the center of the beam due to elastic deformations produced at the time the load is applied.

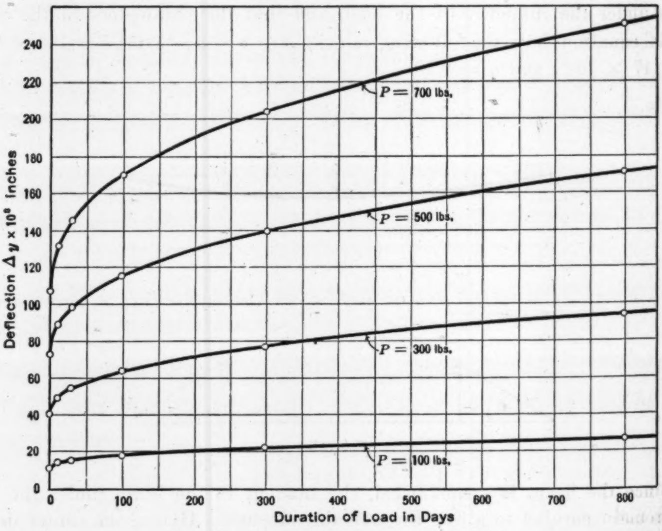


FIG. 19.—PLASTIC DEFLECTION OF BEAM.

Beam Fixed at One End and Supported Unrestrained at the Other End.—Consider the beam, *A B* (Fig. 20 (*a*)). The moment diagram (Fig. 20 (*b*)) is made up of the moment diagram for a simple beam loaded with the force, P, and the diagram due to the fixing of the beam at A.

The mathematics involved in finding the moments is considerably simplified by replacing the moment and shear at A (Fig. 20(c)) by a force, R_L (for convenience in visualizing assumed to act upon the end of a rigid lever arm fastened to the beam at A), applied at the center of the rotation, such that,

$$M_A = R_L\,(a - x') \text{ and } V_A = R_L$$

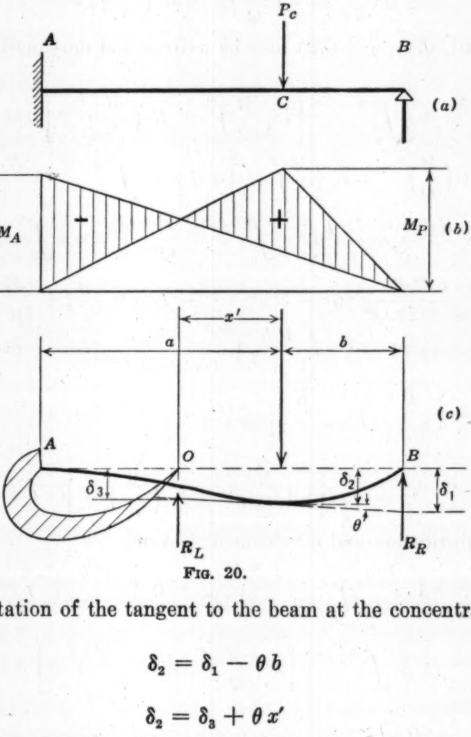


FIG. 20.

If θ is the rotation of the tangent to the beam at the concentrated load, P, by geometry,

$$\delta_2 = \delta_1 - \theta\, b$$

and,

$$\delta_2 = \delta_3 + \theta\, x'$$

Subtracting and re-arranging terms,

$$\delta_1 - \delta_3 - (x' + b)\,\theta = 0 \qquad (68)$$

By statics,

$$R_L = \frac{P\,b}{x' + b};\text{ and } R_R = \frac{P\,x'}{x' + b} \qquad (69)$$

Since δ_3 is the deflection of O away from the tangent at O,

$$\delta_3 = \int_0^{(a-x')} \frac{d\,\phi}{d\,s}\,x\,d\,x + \int_0^{x'} \frac{d\,\phi}{d\,s}\,x\,d\,x \qquad (70)$$

Similarly, δ_1 is the deflection of B away from the tangent at C, hence,

$$\delta_1 = \int_0^{b} \frac{d\,\phi}{d\,s}\,x\,d\,x \qquad (71)$$

The slope of the tangent at C is,

$$\theta = \int_{x'}^{(a-x')} \frac{d\,\phi}{d\,s}\, d\,x \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (72)$$

In Equations (70) and (72) the center of co-ordinates is at O; in Equation (71), B is the center of co-ordinates.

Since,

$$\frac{d\,\phi}{d\,s} = k\left(\frac{M}{Q}\right)^m + K\left(\frac{M}{S}\right)^p t^q$$

Equations (70), (71), and (72) may be written and developed, respectively, as follows:

$$\delta_3 = \int_0^{(a-x')} \left[k\left(\frac{M}{Q}\right)^m + K\left(\frac{M}{S}\right)^p t^q\right] x\,d\,x$$

$$+ \int_0^{x'} \left[k\left(\frac{M}{Q}\right)^m + K\left(\frac{M}{S}\right)^p t^q\right] x\,d\,x = \int_0^{(a-x')} k\,\frac{{R_L}^m\, x^{(m+1)}}{Q^m}\, d\,x$$

$$+ \int_0^{(a-x')} K\,\frac{{R_L}^p\, x^{(p+1)}}{S^p}\, t^q\, d\,x + \int_0^{x'} k\,\frac{{R_L}^m\, x^{(m+1)}}{Q^m}\, d\,x + \int_0^{x'} K\,\frac{{R_L}^p\, x^{(p+1)}}{S^p}\, t^q\, d\,x$$

$$= \frac{k\,{R_L}^m}{(m+2)\,Q^m}\left[(a-x')^{(m+2)} + (x')^{(m+2)}\right] + \frac{K\,{R_L}^p\, t^q}{(p+2)\,S^p}$$

$$\left[(a-x')^{(p+2)} + (x')^{(p+2)}\right] = \left(\frac{P\,b}{x'+b}\right)^m \frac{k}{(m+2)\,Q^m}\left[(a-x')^{(m+2)} + (x')^{(m+2)}\right]$$

$$+ \left(\frac{P\,b}{x'+b}\right)^p \frac{K\,t^q}{(p+2)\,S^p}\left[(a-x')^{(p+2)} + (x')^{(p+2)}\right] \ldots\ldots (73)$$

also,

$$\delta_1 = \int_0^{b} \left[k\left(\frac{M}{Q}\right)^m + K\left(\frac{M}{S}\right)^p t^q\right] x\,d\,x$$

by similar substitution and development becomes,

$$\delta_1 = k\left(\frac{P\,b}{x'+b}\right)^m \frac{(x')^m\, b^2}{(m+2)\,Q^m} + K\left(\frac{P\,b}{x'+b}\right)^p \frac{(x')^p\, b^2\, t^q}{(p+2)\,S^p} \ldots\ldots (74)$$

Similarly,

$$\theta = \int_{x'}^{(a-x')} \left[k\left(\frac{M}{Q}\right)^m + K\left(\frac{M}{S}\right)^p t^q\right] d\,x$$

which becomes,

$$\theta = \left(\frac{P\,b}{x'+b}\right)^m \frac{k}{(m+1)\,Q^m}\left[(a-x')^{(m+1)} - (x')^{(m+1)}\right]$$

$$+ \left(\frac{P\,b}{x'+b}\right)^p \frac{K\,t^q}{(p+1)\,S^p}\left[(a-x')^{(p+1)} - (x')^{(p+1)}\right] \ldots\ldots (75)$$

By substituting the values from Equations (73), (74), and (75) into Equations (68), and collecting terms,

$$k\left[\frac{P\,b}{(x'+b)\,Q}\right]^m \left\{\frac{(x')^m\, b^2}{(m+2)} - \frac{\left[(a-x')^{m+2} + (x')^{m+2}\right]}{(m+2)}\right.$$

$$\left. - \frac{x'+b}{m+1}\left[(a-x')^{(m+1)} - (x')^{(m+1)}\right]\right\} + K\,t^q\left[\frac{P\,b}{(x'+b)\,S}\right]^p \left\{\frac{(x')^p\, b^2}{(p+2)}\right.$$

$$\left. - \frac{\left[(a-x')^{p+2} + (x')^{p+2}\right]}{(p+2)} - \frac{x+b}{p+2}\left[(a-x')^{p+1} - (x')^{p+1}\right]\right\} = 0 \ldots (76)$$

At the beginning of a loaded condition there is no plastic flow; $t = 0$, and hence,

$$\frac{(x')^m b^2}{(m+2)} - \frac{[(a-x')^{m+2} + (x')^{m+2}]}{(m+2)}$$
$$+ \frac{x'+b}{(m+1)} [(a-x')^{m+1} - (x')^{m+1}] = 0 \ldots\ldots\ldots\ldots (77)$$

Special Case, in Which $m = 1$ and $a = b = \frac{1}{2} l$.—This is the condition of a beam of homogeneous material following Hooke's law, and loaded at mid-span. The beam is indicated in Fig. 21(*a*).

By the ordinary theory of elasticity, the statical moment of the area under the moment curve about B (Fig. 21(*b*)) is equal to the deflection of B away from the tangent at A, hence, by simple statics, it will be found that,

$$x' = \frac{5}{22} l$$

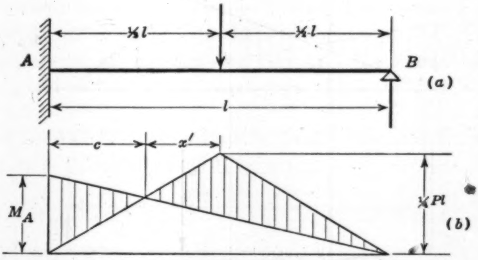


FIG. 21.

A test of Equation (77) is obtained by substituting into the equation the values of c and x' as obtained from the ordinary theory of elasticity. The first member of the equation reduces to zero, thereby verifying the derived equation.

Special Case, in Which $m = 1.3$, and $a = b = \frac{1}{2} l$.—This is an exaggerated case for concrete in which Hooke's law does not apply. The location of the point of contraflexure will be assumed to be the same as that found by the theory of elasticity (stress is proportional to strain). A substitution of the assumed values, $m = 1.3$, into Equation (77) again reduces the expression practically to an identity.

Thus, Equation (77) is satisfied for values of m other than 1, without appreciably moving the point of contraflexure. Hence, one might conclude that the moments in the beam are not affected by static loads, although the modulus of elasticity of the material varies with the stress.

Further, an analysis of the first term of Equation (76) indicates that the flow of the material with the lapse of time does not change the moment distribution in the beam. It will be observed that both members of the first term contain factors similar to the first term of Equation (77). Hence, it may be concluded that Equation (76) will be satisfied by a value of x' which would be obtained by the ordinary theory of elasticity in which Hooke's law is assumed to be correct. This conclusion, of course, is on the basis of a homogeneous material.

Although the distribution of moments in the beam remains the same throughout any time interval, the distribution of stresses and the configuration of the beam will constantly change as the time interval increases.

Beam Fixed at One End, Supported Unrestrained at the Other End, and Free Support Allowed to Settle a Distance, Δy.—The support at B, in Fig. 22(a), is assumed to settle an arbitrary amount, Δy, when the load, P, is applied.

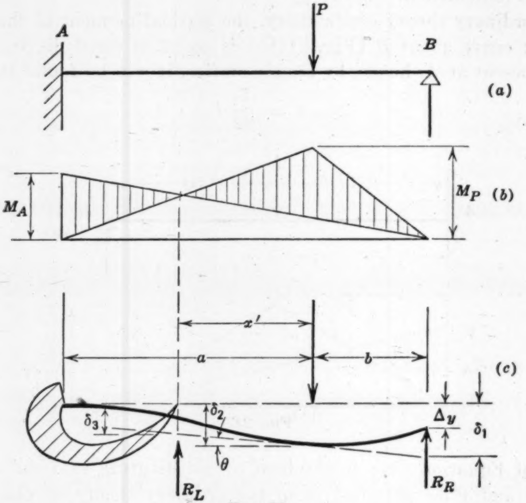


FIG. 22.

Equation (74) then takes the form,

$$\delta_1 = k\left(\frac{P\,b}{x' + b}\right)^m \frac{(x')^m\,b^2}{(m + 2)\,Q^m} + K\left(\frac{P\,b}{x' + b}\right)^p \frac{(x')^p\,b^2\,t^q}{(p + 2)\,S^p} + \Delta\,y$$

This results in a modification of Equation (76), which then may be written as,

$$t^q = \frac{-\,k\left[\dfrac{P\,b}{(x' + b)\,Q}\right]^m \left\{\dfrac{(x')^m\,b^2}{(m+2)} - \dfrac{[(a - x')^{m+2} + (x')^{m+2}]}{(m + 2)} - \dfrac{x' + b}{(m + 1)}\,[(a - x')^{m+1} - (x')^{m+1}]\right\} - \Delta\,y}{K\left[\dfrac{P\,b}{(x' + b)\,S}\right]^p \left\{\dfrac{(x')^p\,b^2}{(p + 2)} - \dfrac{[(a - x')^{p+2} + (x')^{p+2}]}{(p + 2)} - \dfrac{(x' + b)}{(p + 1)}\,[(a - x')^{p+1} - (x')^{p+1}]\right\}} \quad \ldots (78)$$

A scrutiny of Equation (78) immediately discloses that the distribution of moments in the beam changes with the time. A state of equilibrium is approached as the time interval, t, becomes very large.

When x' in Equation (78) becomes equal to the value of x' which satisfies Equation (76), the denominator of the second member of Equation (78) becomes zero; hence, t becomes infinite. This indicates that, with a lapse of time, the moment distribution in the beam approaches that of the beam without settlement of the support.

Immediately after the load, P, is applied with resulting settlement of support, $t = 0$; hence, Equation (78) takes the form,

$$k\left[\frac{P\,b}{(x'+b)\,Q}\right]^m \left\{\frac{(x')^m\,b^2}{(m+2)} - \frac{[(a-x')^{(m+2)} + (x')^{(m+2)}]}{(m+2)} - \frac{x'+b}{(m+1)}\,[(a-x')^{(m+1)} - (x')^{(m+1)}]\right\} + \varDelta\,y = 0 \ldots\ldots (79)$$

The distribution of moments then varies as a function of the time interval. The limiting values of x' will be the value just after loading and settling of the support and the value for the conditions of no settlement of the support. The equations have been derived assuming a statically determinate condition, which, of course, is not directly applicable to indeterminate structures. That is, the equations are so set up that one may find the change in shape of the structure at the end of any time interval provided the moment distribution is known.

Special Case in Which Hooke's Law Is True.—Assuming that Hooke's law applies, that is, $m = 1$, Equation (79) becomes,

$$\varDelta\,y = \frac{P\,b}{(x'+b)\,E\,I}\left\{\frac{x'\,b^2}{3} - \frac{1}{3}\,[(a-x')^3 + (x')^3] - \frac{1}{2}\,(x'+b)\,[(a-x')^2 - (x')^2]\right\} \ldots\ldots\ldots\ldots\ldots\ldots (80)$$

Numerical Case.—As a simple case for the study of the effects of plastic flow, consider the beam in Fig. 23. The following values will be considered to satisfy the properties of the concrete in the exponential equations: $m = 1.15$; $p = 1.30$; $q = 0.40$; $K = 36 \times 10^{-9}$; and $k = 182 \times 10^{-9}$.

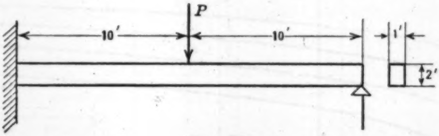


FIG. 23.

Assume that the right support settles at the instant of applying the load such that the right reaction is zero and the beam is just in contact with the support. In Equation (78) x' will be equal to zero at the beginning of the loaded condition. Hence, the settlement of the support will be,

$$\varDelta\,y = k\left(\frac{P}{Q}\right)^m \left[\frac{a^{(m+2)}}{m+2} + \frac{b\,a^{(m+1)}}{m+1}\right]$$

or, substituting the numerical values,

$$\varDelta\,y = P^{1.15} \times 12 \times 10^{-6} \text{ in.}$$

Assume, now, that plastic flow takes place. Deformation will take place in the part of the beam that is subject to stress, and this will result in load being transferred to the right reaction which, it is assumed, is fixed in location, that is, it does not settle further.

Considering a given location of the point of contraflexure in the beam, a study will be made to determine the time interval required for the beam to assume a conformation which will satisfy its end conditions. Thus, when a load, P, is applied at the center of the beam and the deflection at the left support is $\Delta y = P^{1.15} \times 12 \times 10^{-6}$, the point of contraflexure might be considered to be directly under the load, since the moment is zero in the right half of the beam.

If the point of inflection is considered to be a distance, x', to the left of the load, the time required for the slope of the beam at the left support to reach its normal condition may be computed by Equation (78), and would be,

$$t = \left\{ P^{(m-p)} \frac{-k S^p \left\{ \left(\frac{b}{x'+b}\right)^m \left[\frac{(x')^m b^2}{m+2} - \frac{[(a-x')^{m+2} + (x')^{m+2}]}{m+2} - \frac{x'+b}{m+1} [(a-x')^{m+1} - (x')^{m+1}] \right] + \left[\frac{a^{(m+2)}}{m+2} + \frac{b\,a^{(m+1)}}{m+1} \right] \right\}}{K Q^m \left(\frac{b}{x'+b}\right)^p \left\{ \frac{(x')^p b^2}{p+2} - \frac{[(a-x')^{p+2} + (x')^{p+2}]}{p+2} - \frac{x'+b}{p+1} [(a-x')^{p+1} - (x')^{p+1}] \right\}} \right\}^{\frac{1}{q}} \quad (81)$$

Equation (81) has been solved by substituting various numerical values for the distance, x', and for the load, P. All computations for this analysis were made on an 8-in. slide-rule. A graphical representation of the results obtained by studying the movement of the point of contraflexure is shown on Fig. 24. Curves are plotted for various values of the concentrated loads, P.

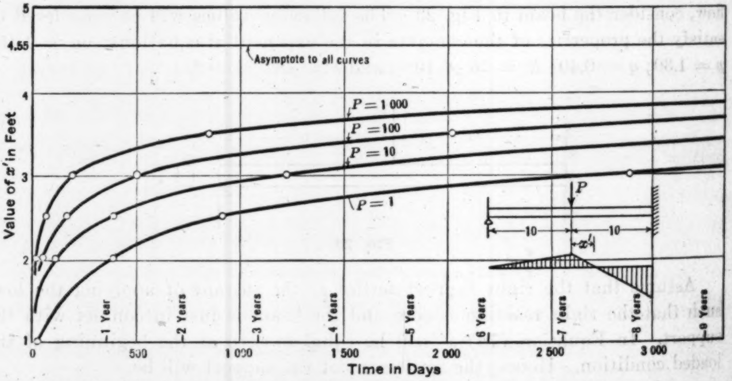


FIG. 24.—MOVEMENT OF POINT OF CENTER OF FLEXURE FOR CONCENTRATED LOAD.

A load of 1 000 lb. produces an extreme fiber stress about equal to the maximum tensile strength obtained in concrete.

When the point of contraflexure approaches a location 4.55 ft. from the center of the beam, the numerator in the second member of Equation (81) becomes zero and the time interval becomes infinite. This "critical point" is at the point of contraflexure which would be found for the condition of pure elastic deformation and no settlement of the support. Fig. 24 shows a line through the critical point as the asymptote to all curves indicating the movement of the point of contraflexure.

The variation of moment distribution as x' changes may readily be computed from simple static and geometric relations in the moment diagram. Curves have been plotted showing the moment redistribution in the beam in which the support has settled and plastic flow has occurred. Fig. 25 shows the curves considering, respectively, two different amounts of settlement of the support, A, and the load to be 1 000 lb.

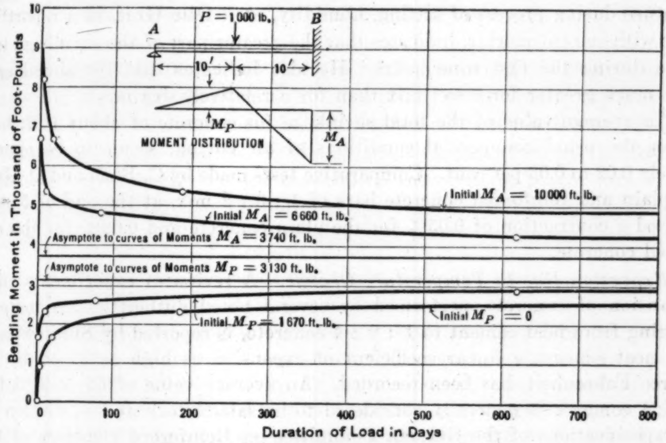


FIG. 25.—MOMENT DISTRIBUTION IN BEAM.

The graphs for the rate of change of the moment curve and point of contraflexure, of course, are subject to the criticism that the gradual changes in the moment curve have not been considered. Such progressive change can be accounted for by successive approximations and would result in the beam approaching a state of equilibrium more rapidly than is indicated by the diagrams. However, this refinement is not warranted with the present status of knowledge of the plastic properties of concrete.

V.—ANALYSIS OF ARCHES

Choice of Parameters in the Proposed Theories.—Since no experimental studies of concrete have been made with the particular intention of fitting the results into theories such as the writer has proposed, it would be very strange, to say the least, to find many data which can be used directly to define numerically the parameters in the proposed equations. In general, the duration of the time-yield tests has been too short to give precise results. Usually, especially in foreign experiments, plastic flow has not been distinguished from shrinkage. In their studies of arches, the French simply combine the plastic flow and shrinkage into one effect, calling it *retrait*.

Nevertheless, since a study shows that plastic deformations on the one hand, and shrinkage and temperature deformations on the other, produce very different effects on the stresses in a structure, it becomes quite important that these properties be differentiated.

Shrinkage of Concrete.—Shrinkage is distinguished from plastic flow, in that it is a change in dimensions independent of external forces acting on the specimen.

The shrinkage of concrete varies within such wide limits that it is practically impossible, at present, to assign a definite value or law to its coefficient. Among the many factors[16] influencing this property are: The composition of the cement, quality of aggregate, proportioning of mix, wetness of mix, temperature during process of setting, humidity, etc. Otto Graf, in a report[17] on tests with cement mortar, indicates that the greater part of the shrinkage takes place during the first nine hours. He also indicates that the shrinkage is very much greater for a wet mix than for a relatively dry mix.

An average value of the total shrinkage for concrete of about 1:2:4 mix under the usual commercial conditions of air curing, seems to be approximately 0.02 to 0.03 per cent. Comparative tests made by C. Bach and O. Graf[18] on plain and reinforced concrete bars of 1 : 3 : 2 mix, at the end of 5 years, showed a contraction of 0.05% for the plain concrete and 0.02% for the reinforced concrete.

Expansion Due to Temperature Change.—A review of experiments on the expansion of concrete, performed by several noted authorities on materials ranging from neat cement to 1 : 2 : 4 concrete, is reported by Shitkewitsch.[19] For neat cement, a linear coefficient of expansion as high as 81×10^{-7} per degree Fahrenheit, has been recorded. An average value of 55×10^{-7} for a 1:2:4 concrete is generally considered to be satisfactory and is written into the specifications of the German Committee on Reinforced Concrete of 1925, and also into the specifications of the Belgian Standards Association of 1923. This value is corroborated in experiments by W. D. Pence, M. Am. Soc. C. E.[20]

Within the variation of the ordinary atmospheric temperatures, the coefficient of expansion is practically constant. Indeed, for the study of stresses in concrete arches, the coefficient of conductivity through the concrete is of tremendously greater importance than the variation found in the coefficient of expansion.

Choice of Parameters for Exponential Formulas Defining Plastic Deformation in Arches.—Although the exponents, p and q, may vary considerably with concretes of various ages and qualities, the writer believes that they might be predetermined with a fair degree of precision by a systematic series of experiments. In a concrete which is not more than 2 weeks old, it appears that q might be as low as 0.15, while p might reach 2.0, or more. Concrete of a 1:2:4 mix subjected to a constant load beginning at about the age of

---

[16] "Einflüsse auf Beton," A. Kleinlogel, p. 314, Berlin, 1925.

[17] *Beton und Eisen*, 1921-22.

[18] "Armierter Beton," Bach and Graf, 1909.

[19] "Monolität d. Betonbauten," N. A. Shitkewitsch, *Forscherarbeiten* a. d. Gebiete d. Eisenbetons, Heft 7, 1906.

[20] *Journal*, West. Soc. of Engrs., Vol. VI, p. 549, 1901.

4 months, would probably deform according to the exponential equation with q and p, respectively, equal to about 0.4 and 1.25.

From observations made on a large number of arches, E. Freyssinet, a French engineer who has developed mechanical methods of caring for rib-shortening effects in arches, gives values[21] for the shortening of a rib over a long period of time. The unit shortenings are listed for various cement contents as follows:

350 kg. per cu. m. (1.56 bbl. per cu. yd.).....	4×10^{-4} to 6×10^{-4}
400 " " " " (1.78 " " " ").....	5×10^{-4} to 7×10^{-4}
450 " " " " (2.00 " " " ").....	6×10^{-4} to 8×10^{-4}

A curve representing the shortening over a long period of time would fluctuate above and below a mean which would slowly approach zero. The fluctuation, of course, is dependent on the temperature and humidity variations.

Assuming that, in the arches observed by Freyssinet, the shrinkage of the concrete due to setting was about 0.0003, the plastic shortening of the arch ribs made of the richer concrete would be about $0.0007 - 0.0003 = 0.0004$. Hence, for an analysis of an arch in which the centering is moved within about 4 months after pouring the rib, the exponential equation for plastic flow would probably approximately take the form,

$$\epsilon = 17 \times 10^{-9} \sigma^{1.25} t^{0.4} \ldots\ldots\ldots\ldots\ldots\ldots (82)$$

in which, σ is the stress, in pounds per square inch, and t is the time, in days, after striking the centering. This would result in a plastic flow rib-shortening of about 0.00035 in 2 years, if the average dead load stress in the arch is 350 lb. per sq. in. According to Equation (82), it would require almost 10 years more to double this rib-shortening. It is probable that the exponential equation would take a somewhat different form as the concrete becomes older; however, no definite determination can be made from the experimental data now available.

The numerical parameters given in Equation (82) will not be strictly adhered to, in the studies which follow, but the constants chosen will be based on considerations similar to those just presented.

DESCRIPTION OF ARCHES ANALYZED

Description of Arch No. 1.—Arch No. 1 is a symmetrical solid arch barrel having a rise ratio of 1:4. The center line of the arch is a parabola with a span of 140 ft., and a rise of 35 ft. from springing to crown. The thickness of the rib is 2 ft. at the crown and varies as the square of the secant of the angle between the tangent to the arch center line and the horizontal. Various assumptions have been made with regard to the properties of the material composing the arch. These variations will be explained when they are introduced into the discussion and the varied analyses are carried to completion. As far as possible, all variations are based on rational grounds which are borne out by experiment.

---

[21] "Le Pont de Villeneuve-sur-Lot," M. Freyssinet, *Le Genie Civil*, v. 79, p. 126, August, 1921.

Description of Arch No. 2.—Arch No. 2 is practically the same as Arch No. 1, except that the rise ratio is 1:8 instead of 1:4.

General Analysis of the Arch.—The arch rib (Fig. 26) is divided into twenty equal horizontal lengths, $\Delta x = 7$ ft. A point half way between the ends of each voussoir was located and numbered, the numbering being from left to right. Thus, Arch-Point No. 1 is a point on the center line of the arch, $3\frac{1}{2}$ ft. horizontally to the right of the left springing; Point No. 2 is $10\frac{1}{2}$ ft. horizontally to the right, etc. All distances are measured in feet.

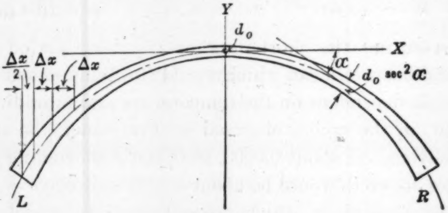


FIG. 26.

Deformations of the rib were found by computing the rate of angle change, $\frac{d\,\phi}{d\,s}$, and rate of rib-shortening, $\frac{d\,\xi}{d\,s}$, respectively, at each arch point, and assuming the total deformation within a voussoir to be concentrated at the corresponding arch point. Whenever possible, constant factors have been omitted from the computations, and introduced only where necessary to obtain relative results. In cases where logarithms were needed for numerical analysis, a constant factor was sometimes introduced to simplify the computation.

The center line of the parabolic arch with respect to the crown as the center of a rectangular co-ordinate system, is given by the equation,

$$y = \frac{2\,k}{x^9} \qquad \text{(83)}$$

in which, k is a parameter equal to the distance from the focus to the directrix of the parabola, or twice the distance from the focus to the crown or apex of the parabola. For the arch having a rise ratio of 1:4 and a 140-ft. span, $k = 70$ ft. The co-ordinates of the arch points were computed by Equation (83).

The slope of the tangent to the center line of the rib at any point is the first derivative of y with respect to x, hence,

$$\tan \alpha = \frac{d\,y}{d\,x} = -\frac{x}{k}$$

The cosine of the angle, α, was read from a table of trigonometric functions. The thickness of the rib at any point is,

$$d = 2\,c = \frac{2\,c_0}{\cos^2 \alpha}$$

in which, c_0 is equal to one-half the crown thickness. Hence, since the thickness at the crown of the arch is 2, the vertical location of the neutral point, according to the ordinary theory of elasticity is,

$$y_c' = \frac{\sum\limits_R^L \frac{y_c\, \Delta x}{E\, I \cos \alpha}}{\sum\limits_R^L \frac{\Delta x}{E\, I \cos \alpha}} = \frac{\sum\limits_R^L y_c \cos^5 \alpha\, \Delta x}{\sum\limits_R^L \cos^5 \alpha\, \Delta x}$$

in which, y_c is the distance from the crown of the arch to an arch point. The distance from the neutral point to an arch point is then,

$$y = y_c - y_c'$$

Table 1 contains the computations for the location of the neutral point and the co-ordinates of the arch points. Then, $y_c' = \frac{41.21}{5.893} = 7.00$.

STUDIES OF THE EFFECTS OF A VARIABLE MODULUS OF ELASTICITY ON THE ACTION OF ARCHES

Elastic Arch Uniformly Loaded and Rib-Shortening for All Causes.—Consider a parabolic arch of the form, Arch No. 1. Fig. 27 shows the center line or axis of the arch, $L\,R$, as a solid line.

Suppose the arch to be uniformly loaded. Considering only flexural deformations in the arch, neglecting rib-shortening entirely, the line of pressure will be a parabola coinciding with the axis of the rib. Taking into account all causes except plastic flow, that is, elastic rib-shortening, temperature change, and shrinkage, there results a rid-shortening thrust, H_{RS}, which must act through the neutral point. Hence, at any point in the arch, the bending moment will be $M = H_{RS}\, y$.

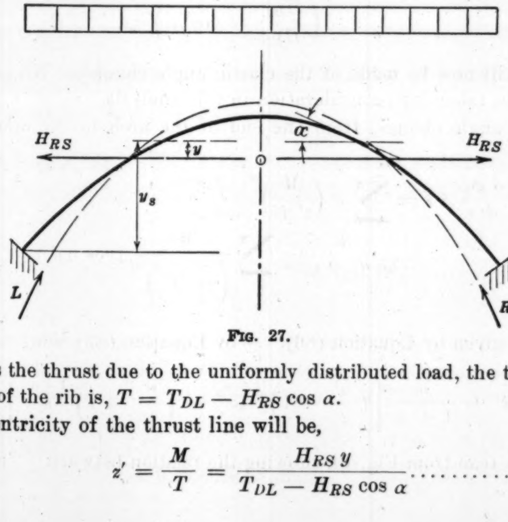


FIG. 27.

If T_{DL} is the thrust due to the uniformly distributed load, the total thrust at any section of the rib is, $T = T_{DL} - H_{RS} \cos \alpha$.

The eccentricity of the thrust line will be,

$$z' = \frac{M}{T} = \frac{H_{RS}\, y}{T_{DL} - H_{RS} \cos \alpha} \qquad (84)$$

or,

$$z' = \frac{y}{\dfrac{T_{DL}}{H_{RS}} - \cos \alpha}$$

Since the arch is parabolic and uniformly loaded, the dead load thrust may be written in terms of the horizontal thrust,

$$T_{DL} = \frac{H_{DL}}{\cos \alpha} \quad \text{.........................(85)}$$

TABLE 1.—COMPUTATIONS FOR THE LOCATION OF THE NEUTRAL POINT, AND CO-ORDINATES OF THE ARCH POINTS, FOR ARCH NO. 1

Point.	Δx.	x.	$\frac{dy}{dx}$.	cos a.	$\cos^5 a$.	$y_c = \frac{x^2}{2k}$.	$y_c \cos^5 a$.	y.
1	7.0	66.5	0.950	0.725	0.2005	31.60	6.33	−24.60
2	7.0	59.5	0.850	0.762	0.256	25.30	6.47	−18.30
3	7.0	52.5	0.750	0.800	0.328	19.68	6.45	−12.68
4	7.0	45.5	0.650	0.838	0.413	14.80	6.11	− 7.80
5	7.0	38.5	0.550	0.876	0.518	10.60	5.49	− 3.60
6	7.0	31.5	0.450	0.912	0.632	7.07	4.46	− 0.07
7	7.0	24.5	0.350	0.944	0.748	4.28	3.20	3.72
8	7.0	17.5	0.250	0.970	0.856	2.18	1.87	4.82
9	7.0	10.5	0.150	0.989	0.945	0.78	0.74	6.22
10	7.0	3.5	0.050	0.999	0.997	0.09	0.09	6.91
Σ	...	....	.....	.....	5.893	....	41.21	....

Taking into consideration that the thickness of the rib at any section is,

$$2\,c = 2\,c_0 \sec^2 \alpha \quad \text{.........................(86)}$$

Equations (84), (85), and (86) may be combined to give the relation,

$$\frac{z'}{c} = \frac{y \cos^3 \alpha}{\left(\dfrac{H_{DL}}{H_{RS}} - \cos^2 \alpha\right) c_0} \quad \text{..................(87)}$$

A study will now be made of the elastic angle changes. No plastic deformations will be taken into consideration in this analysis.

The total angle change, from one end of the arch to the other, is represented by,

$$\sum \frac{d\,\phi_E}{d\,s}\,\Delta\,s = \sum k \left(\frac{M}{z'\,U}\right)^m \frac{\Delta\,x}{\cos \alpha}$$

$$= \frac{k}{b^m\,(c_0)^{1+2m}} \sum \frac{M^m}{\left(\dfrac{z'}{c}\,U'\right)^m} (\cos \alpha)^{1+4m}\,\Delta\,x \text{....(88)}$$

in which, c is given by Equation (86), $\frac{z'}{c}$ by Equation (87), and,

$$U' = \frac{m}{1+m}\left[\left(\frac{u'}{c}+1\right)^{\frac{(1+m)}{m}} - \left(\frac{u'}{c}-1\right)^{\frac{(1+m)}{m}}\right]$$

which may be read from Fig. 10, showing the relation between $\frac{z'}{c}$ and U'.

Similarly, the horizontal movement of one springing relative to the other will be,

$$\Delta l = \sum \frac{d\,\phi_E}{d\,s}\, y_s\, \Delta\, s = \frac{k}{b^m\,(c_0)^{(1+2m)}} \sum \frac{M^m}{\left(\frac{z'}{c}\, U'\right)^m} (\cos\alpha)^{1+4m}\, y_s\, \Delta\, x \,..(89)$$

in which, y_s is the vertical distance from a line joining the springings to an arch point.

Assume that the rib-shortening thrust, H_{RS}, acts through the neutral point corresponding to the ordinary theory of elasticity. Table 2 indicates the computations for the eccentricity of the thrust line, the angle changes, and the relative horizontal movement of the springings, for the ratio, $\frac{H_{DL}}{H_{RS}} = 25$ and $m = 1$. The relations obtained from the proposed theory are used in making the analysis. For the special case in which m is unity, Equations (88) and (89), respectively, become,

$$\sum \frac{d\,\phi_E}{d\,s}\, \Delta\, s = \frac{k}{b\,(c_0)^3} \sum \frac{M}{\frac{z'}{c}\, U'} \cos^5\alpha\, \Delta\, x \ldots\ldots\ldots\ldots(90)$$

and,

$$\Delta\, l = \frac{k}{b\,(c_0)^3} \sum \frac{M}{\frac{z'}{c}\, U'} \cos^5\alpha\, y_s\, \Delta\, x \ldots\ldots\ldots\ldots\ldots(91)$$

The elastic rib-shortening has not been included in $\Delta\, l$ in either Equation (89) or Equation (91). For convenience, the reciprocal of the factors, $\frac{k}{b^m\,(c_0)^{(1+2m)}}$, has been called f in Tables 2 and 3. From Table 2,

$$\Delta\, l = -\frac{183}{144} \times 10^{-9}\,(961.2 - 356.7) \times 7 \times 2 \times H_{RS} = 0.00001075\, H_{RS}$$

Table 3 gives the angle changes and relative movement of the springings for the ratio, $\frac{H_{DL}}{H_{RS}} = 25$ and $m = 1.3$. Thus, Equation (88) and Equation (89), respectively, take the forms,

$$\sum \frac{d\,\phi_E}{d\,s}\, \Delta\, s = \frac{k}{b^{1.3}\, c_0^{\,3.6}}\, 10^{-0.2} \sum \left(\frac{M}{\frac{z'}{c}\, U'}\right)^{1.3} (\cos\alpha)^6 \times (10\cos\alpha)^{0.2}\, \Delta\, x \,..(92)$$

and,

$$\Delta\, l = \frac{k}{b^{1.3}\, c_0^{\,3.6}}\, 10^{-0.2} \sum \left(\frac{M}{\frac{z'}{c}\, U'}\right)^{1.3} (\cos\alpha)^6 \times (10\cos\alpha)^{0.2}\, y_s\, \Delta\, x \,..(93)$$

The computations have been carried out on the assumption that the neutral point, considering the modulus of elasticity a variable, coincides with the neutral point in which $m = 1$. Using the results of Table 3,

$$\Delta\, l = \frac{183}{(144)^{1.3}} \times 10^{-9}\,(4\,043 - 1\,465)\, 10^{-0.2} \times 7 \times 2 \times (H_{RS})^{1.3}$$
$$= 0.00000648\,(H_{RS})^{1.3}$$

TABLE 2.—COMPUTATIONS FOR THE ECCENTRICITY OF THE THRUST LINE, ANGLE CHANGES, AND RELATIVE HORIZONTAL MOVEMENT OF SPRINGING. ARCH UNIFORMLY LOADED AND RIB-SHORTENING DUE TO ALL CAUSES

$$\left(m = 1;\ \frac{H_{DL}}{H_{RS}} = 25.\right)$$

Point.	cos α. (1)	y. (2)	cos² α. (3)	cos³ α. (4)	cos⁵ α. (5)	$y \cos^3 \alpha$. (6)	$\frac{H_{DL}}{H_{RS}} - \cos^2 \alpha$. (7)	$\frac{z'}{c}$. (8)	$\frac{1}{\frac{z'}{c} U'}$. (9)	$f \frac{d\phi}{ds}$ = Column (2) × Column (5) × Column (9). (10)		y_s. (11)	$f \frac{d\phi}{ds} y_s$ = Column (10) × Column (11). (12)	
1	0.725	−24.60	0.526	0.381	0.2005	−9.37	24.474	−0.3830	1.5	+7.28	......	3.40	+ 25.1	....
2	0.762	−18.30	0.580	0.442	0.256	−8.08	24.420	−0.3340	1.5	+7.03	......	9.70	+ 68.2	....
3	0.800	−12.68	0.640	0.512	0.328	−6.48	24.360	−0.2665	1.5	+6.23	......	15.32	+ 95.5	....
4	0.838	− 7.80	0.702	0.588	0.413	−4.58	24.298	−0.1890	1.5	+4.83	......	20.20	+ 97.7	....
5	0.876	− 3.60	0.768	0.673	0.518	−2.39	24.232	−0.0985	1.5	+2.80	......	24.40	+ 68.3	....
6	0.912	− 0.07	0.832	0.759	0.632	−0.05	24.178	−0.0219	1.5	+0.07	......	27.93	+ 1.9	....
7	0.944	+ 2.72	0.890	0.840	0.748	+2.29	24.110	+0.0950	1.5	......	− 3.06	30.72	....	− 94.0
8	0.970	+ 4.82	0.940	0.912	0.856	+4.40	24.060	+0.1830	1.5	......	− 6.18	32.82	....	−203.2
9	0.989	+ 6.22	0.978	0.967	0.945	+6.01	24.021	+0.2500	1.5	......	− 8.82	34.22	....	−302.0
10	0.999	+ 6.91	0.998	0.997	0.995	+0.89	24.002	+0.2870	1.5	......	−10.38	34.91	....	−362.0
Σ	.....	......	......	......	......	.....	......	......	...	+28.34	−28.44	.....	+356.7	−961.2

TABLE 3.—COMPUTATIONS FOR THE ANGLE CHANGES AND THE RELATIVE HORIZONTAL MOVEMENT OF THE SPRINGINGS. ARCH UNIFORMLY LOADED AND RIB-SHORTENING DUE TO ALL CAUSES

$$\left(m = 1.3;\ \frac{H_{DL}}{H_{RS}} = 25.\right)$$

Point.	y. (1)	10 cos α. (2)	log Column (2). (3)	0.2 × Column (3). (4)	$(10 \cos \alpha)^{0.2}$. (5)	$\cos^6 \alpha$. (6)	Column (5) × Column (6). (7)	$\frac{z'}{c}$. (8)	$\frac{1}{U'}$. (9)	$\frac{M}{\frac{z'}{c} U'}$. (10)	log Column (10). (11)	Column (11) × 1.3. (12)	$\frac{M^{1.3}}{\left(\frac{z'}{c} U'\right)^{1.3}}$. (13)	$f\frac{d\phi}{ds}$ = Column (13) × Column (7). (14)		y_s. (15)	$f\frac{d\phi}{ds}y_s$. (16)		$\frac{u'}{c}$. (17)	$f\frac{d\xi}{ds} = \frac{\text{Column (14)} \times \text{Column (17)}}{\cos \alpha}$. (18)
1	−24.60	7.25	0.860	0.1720	1.482	0.145	0.215	−0.384	0.680	43.5	1.638	2.130	135.0	+29.0	......	3.40	+ 98	......	0.70	28.0
2	−18.30	7.62	0.882	0.1764	1.491	0.195	0.291	−0.334	0.605	33.2	1.521	1.980	95.0	+27.7	......	9.70	+ 268	......	0.85	30.9
3	−12.68	8.00	0.908	0.1806	1.515	0.262	0.397	−0.267	0.507	24.0	1.380	1.796	62.5	+24.8	......	15.32	+ 380	......	1.03	32.0
4	− 7.80	8.38	0.923	0.1846	1.529	0.346	0.529	−0.189	0.390	16.1	1.207	1.569	37.1	+19.7	......	20.20	+ 396	......	1.41	33.2
5	− 3.60	8.76	0.942	0.1882	1.542	0.453	0.698	−0.099	0.260	9.42	0.974	1.266	18.5	+12.9	......	24.40	+ 315	......	2.52	37.0
6	− 0.07	9.12	0.960	0.1920	1.553	0.576	0.895	−0.022	0.117	0.37	.....	.....	0.3	+ 0.3	.....	27.93	+ 8	......	8.20	38.0
7	+ 2.72	9.44	0.975	0.1950	1.567	0.707	1.108	+0.095	0.235	6.88	0.837	1.088	12.3	......	−13.6	30.72	.....	− 418	2.71	38.9
8	+ 4.82	9.70	0.987	0.1974	1.575	0.831	1.309	+0.183	0.373	9.85	0.993	1.291	19.6	......	−25.7	32.82	.....	− 845	1.47	39.1
9	+ 6.22	9.89	0.995	0.1990	1.581	0.936	1.480	+0.250	0.480	11.85	1.074	1.398	25.0	......	−37.0	34.22	.....	−1268	1.10	41.2
10	+ 6.91	9.99	0.999	0.1998	1.584	0.994	1.575	+0.281	0.532	12.80	1.107	1.439	27.5	......	−43.3	34.91	.....	−1512	1.00	43.2
Σ	....	....	.....	.....	.....	.....	.....	....	.....	.....	.....	.....	.....	+114.4	−119.6	.....	+1465	−4043	....	361.5

It is realized, of course, from the experimental data on the elastic properties of concrete that a variable relation of stress to elastic strain in which m is as high as 1.3 is probably never obtained in practice. However, an extreme value of m has been chosen in order that the results may show characteristic trends more prominently. Also, since a value of p of 1.3 in the proposed theory of plasticity is probably about correct for concrete 3 or 4 months old, this value makes a convenient starting point for an analysis in which the plastic properties of the material are considered.

A comparison of Tables 2 and 3 reveals some interesting phenomena. Both sets of computations were made on arches of identical dimensions and loadings, but of materials having very different elastic properties. In the first case (Table 2), Hooke's law was assumed to be true. This condition could have been solved by the ordinary theory of elasticity and the resulting differential angle changes, except for slight errors in precision, would have been identical with those obtained by an application of the proposed theory of elasticity, in which $m = 1$.

In both cases the rib-shortening thrust was considered to act in the same relative location. Although the rate of angle change at corresponding sections in the two arches differs greatly, the total angle change over the arch ring in both cases is practically equal to zero. However, change in span due to the rib-shortening thrust in the two cases varies, that for a constant modulus of elasticity being much less than that for a variable relation of stress to strain. For example, assume a uniformly distributed load to be placed on the arch of such a magnitude that the average compression at the crown is 200 lb. per sq. in. The rib-shortening thrust will be, $H_{RS} = 200 \times 12 \times 24 \div 25 = 2\,305$ lb. Hence, for $m = 1, \Delta l = 0.00001075 \times 2\,305 = 0.0248$ ft., and for $m = 1.3$, $\Delta l = 0.00000648 \times (2\,305)^{1.3} = 0.153$ ft.

As a result, assuming the coefficient of expansion in both arch materials to be constant and equal to each other, one might conclude that if the rib-shortening in the first case was due entirely to a temperature change of 36°, in the second case (a variable, E, being assumed) the temperature change which would cause the same stressed condition would be, $\Delta t^{\circ} = 36^{\circ} \times \frac{0.153}{0.0248} = 222^{\circ}$. This comparison, however, is not entirely fair, because if one were solving an arch problem by the theory of elasticity he would probably choose a value of E which would correspond to the secant modulus at the allowable working stresses, thus resulting in a different comparison. The foregoing comparison is based on the initial tangent modulus in both cases considered. Nevertheless, this analysis does indicate that temperature stresses computed by the ordinary theory of elasticity are incorrect.

ANALYSIS OF ELASTIC ARCH SUBJECTED TO A UNIFORMLY DISTRIBUTED DEAD LOAD AND DEAD LOAD RIB-SHORTENING

Consider Arch No. 1 to be uniformly loaded. If there were no rib-shortening effects, the line of pressure would coincide with the axis of the rib and there would be no stresses due to flexure. However, the rib of the arch shortens under the load and, as a result, flexural stresses arise.

Neglecting all but elastic deformations, which is the case obtained at the instant of striking the forms of a newly constructed arch, the condition that there shall be no horizontal movement of the abutments is defined by,

$$\sum k \left(\frac{M}{z'U}\right)^m y \frac{\Delta x}{\cos\alpha} + \sum k \left(\frac{M}{z'U}\right)^m u' \Delta x = 0 \ldots\ldots\ldots (94)$$

Since the moments are due entirely to the rib-shortening thrust, H_{RS},

$$M = H_{RS}\, y$$

y being the distance from the line of action of the rib-shortening thrust to the arch point considered. Equation (94) may thus be transformed into,

$$(H_{RS})^m k \sum \left(\frac{y}{z'U}\right)^m y \frac{\Delta x}{\cos\alpha} + (H_{RS})^m k \sum \left(\frac{y}{z'U}\right)^m u' \Delta x = 0 \,.. (95)$$

An examination of Equation (95) indicates that the eccentricity of the line of thrust at any point is independent of the intensity of the thrust, for, by dividing the equation by $(H_{RS})^m$, it will be observed that every factor is independent of the stress in the arch. Hence, it may be concluded that H_{RS} varies directly as the intensity of the dead load thrust, regardless of the values assigned to the parameters which define the properties of the material.

The problem now resolves itself into finding a ratio between the rib-shortening thrust and the dead load thrust which will give an eccentricity, z', satisfying Equation (95).

Solution for $m = 1$.—When $m = 1$, which is merely the case of the ordinary theory of elasticity, the problem is most easily solved by transforming Equation (95) into the form,

$$H_{RS} \sum k \frac{y^2}{I} \frac{\Delta x}{\cos\alpha} + \sum k \frac{N}{A} \Delta x = 0 \ldots\ldots\ldots\ldots\ldots (96)$$

but assuming absolute values for force vector,

$$N = -H_{RS} \cos\alpha + \frac{H_{DL}}{\cos\alpha}$$

Substituting this value into Equation (96), and re-arranging the terms,

$$\frac{H_{RS}}{H_{DL}} = \frac{\sum \frac{\Delta x}{A \cos\alpha}}{\sum \frac{y^2}{I} \frac{\Delta x}{\cos\alpha} + \sum \frac{\cos\alpha}{A} \Delta x} \ldots\ldots\ldots\ldots\ldots (97)$$

Table 4 has been computed on the basis of Parabolic Arch No. 1, in which,

$$\left.\begin{aligned} d &= d_0 \sec^2\alpha \\ \text{hence,}\quad I &= \frac{1}{12} b\, d_0^3 \sec^6\alpha \\ \text{and,}\quad A &= b\, d_0 \sec^2\alpha \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (98)$$

Thus, when Equations (98) are substituted into Equation (97), and making $d_0 = 2$,

$$\frac{H_{RS}}{H_{DL}} = \frac{\Sigma \cos\alpha\, \Delta x}{3\, \Sigma\, y^2 \cos^5\alpha\, \Delta x + \Sigma \cos^3\alpha\, \Delta x} \ldots\ldots\ldots\ldots (99)$$

From Table 4,

$$\frac{H_{RS}}{H_{DL}} = \frac{8.815}{3 \times 401.1 + 7.071} = \frac{1}{137.3}$$

The eccentricity of the thrust line at each arch point is given in Table 5.

TABLE 4.—COMPUTATION OF RATIO OF HORIZONTAL THRUSTS FOR RIB-SHORTENING AND UNIFORMLY DISTRIBUTED LOAD ON ARCH NO. 1.

($m = 1$; $d_0 = 2$.)

Point.	Δx.	y.	$\cos \alpha$.	$\cos^3 \alpha$.	$\cos^5 \alpha$.	y^2.	$y^2 \cos^5 \alpha$.
1	7	−24.60	0.725	0.381	0.201	605.0	121.4
2	7	−18.30	0 762	0.442	0.256	335.0	85.8
3	7	−12.68	0.800	0.512	0.328	160.5	52.7
4	7	− 7.80	0.838	0.588	0.413	60.8	25.1
5	7	− 3.60	0.876	0.673	0.518	12.96	6.7
6	7	− 0.07	0.912	0.759	0.632	0.005	0.0
7	7	+ 2.72	0.944	0.840	0.748	7.40	5.5
8	7	4.82	0.970	0.912	0.856	23.2	19.9
9	7	6.22	0.989	0.967	0.945	38.6	36.5
10	7	6.91	0.999	0.997	0.995	47.8	47.5
Σ	..	......	8.815	7.071	......	......	401.1

Solution for $m = 1.3$.—When the value of m is other than unity, a solution becomes more cumbersome. For this case, the most convenient form of Equation (95) seems to be,

$$(H_{RS})^m \sum \frac{k}{b^m\, c^{1+2m}} \frac{y^m}{\left(\frac{z'}{c}\, U'\right)^m} y_s \frac{\Delta x}{\cos \alpha}$$

$$+ (H_{RS})^m \sum \frac{k}{b^m\, c^{2m}} \frac{y^m}{\left(\frac{z'}{c}\, U'\right)^m} \frac{u'}{c}\, \Delta x = 0 \ldots\ldots\ldots\ldots (100)$$

This relation takes the following form for both Arch No. 1 and Arch No. 2,

$$(H_{RS})^m \frac{k}{b^m\, (c_0)^{2m}}\, 10^{-0.2} \left[\frac{1}{c_0} \sum \frac{y^m}{\left(\frac{z'}{c}\, U'\right)^m} y_s\, (10 \cos \alpha)^{0.2} (\cos \alpha)^6\, \Delta x \right.$$

$$\left. + \sum \frac{y^m}{\left(\frac{z'}{c}\, U'\right)^m} \frac{u'}{c}\, (10 \cos \alpha)^{0.2} (\cos \alpha)^5\, \Delta x \right] = 0 \ldots\ldots\ldots (101)$$

The requirement that the abutments shall not rotate is satisfied when,

$$(H_{RS})^m \frac{k}{b^m\, (c_0)^{1+2m}}\, 10^{-0.2} \sum \frac{y^m}{\left(\frac{z'}{c}\, U'\right)^m} (10 \cos \alpha)^{0.2} (\cos \alpha)^6\, \Delta x = 0 \ldots (102)$$

To obtain the correct line of thrust now resolves itself into finding the value of the ratio, $\frac{H_{RS}}{H_{DL}}$, and the location of the line of action of H_{RS}, which will satisfy Equation (101) and Equation (102).

Computations for Arch No. 1, in which $m = 1.3$, are shown on Table 6. The calculations are made on the assumption that the rib-shortening thrust is the same as that found for $m = 1$. It will be observed that the sum of the angle changes are practically equal to zero. However, considerable relative horizontal movement of the abutments is indicated. Consequently, one might conclude that the rib-shortening thrust lies in relatively the same line of action for the cases of constant E and variable E, but that the magnitude of this thrust is the smaller in the case of the variable modulus. The line of action for the rib-shortening thrust, however, lies slightly higher for the case of the variable modulus, this being evidenced by the fact that the angle change in the arch rib portion above the assumed line of action of the rib-shortening thrust is slightly greater than the sum of the angle changes in the portion below. (See Table 6, Column (11).)

TABLE 5.—COMPUTATION FOR ECCENTRICITY, $\frac{z'}{c}$, DUE TO UNIFORMLY DISTRIBUTED LOAD ON ARCH NO. 1.

$$\left(c_0 = 1;\ \frac{H_{DL}}{H_{RS}} = 137.3.\right)$$

Point.	y. (1)	$\cos \alpha$. (2)	$\cos^2 \alpha$. (3)	$\cos^3 \alpha$. (4)	$y \cos^3 \alpha$. (5)	$\frac{H_{DL}}{H_{SR}} - \cos^2 \alpha$. (6)	$\frac{z'}{c} = \frac{\text{Column (5)}}{\text{Column (6)}}$ (7)
1	−24.60	0.725	0.526	0.381	−9.37	136.8	0.0685
2	−18.30	0.762	0.580	0.442	−8.08	136.7	0.0592
3	−12.68	0.800	0.640	0.512	−6.48	136.7	0.0475
4	− 7.80	0.838	0.702	0.588	−4.58	136.6	0.0336
5	− 3.60	0.876	0.768	0.673	−2.39	136.5	0.0175
6	− 0.07	0.912	0.832	0.759	−0.05	136.5	0.00037
7	+ 2.72	0.944	0.890	0.840	+2.29	136.4	0.0168
8	+ 4.82	0.970	0.940	0.912	+4.40	136.4	0.0323
9	+ 6.22	0.989	0.978	0.967	+6.01	136.3	0.0441
10	+ 6.91	0.999	0.998	0.997	+6.89	136.3	0.0509

Analysis for Arch No. 2.—For Arch No. 2 the equation of the center line with reference to the crown as origin becomes, $y = \frac{x^2}{280}$ and $\tan \alpha = \frac{d\,y}{d\,x} = \frac{x}{140}$. Table 7 contains the computations for the location of the elastic neutral points. All distances are measured in feet. The constant factor, $\frac{3}{2}\ \frac{\Delta\ x}{b\,c^3_0}$, has been omitted from all partial elastic weights. Using the results from Table 7, $y' = \frac{41.63}{8.343} = 4.98$.

The ratio of the horizontal thrusts for rib-shortening and uniformly distributed load, and the corresponding eccentricity of the thrust line are calculated in Table 8, Columns (1) to (12), inclusive, on the consideration that m is 1. From Table 8,

$$\frac{H_{RS}}{H_{DL}} = \frac{9.624}{3 \times 199.5 + 8.9} = \frac{1}{63.1}.$$

TABLE 6.—RELATIVE HORIZONTAL MOVEMENT OF CANTILEVERS TERMINATING AT NEUTRAL POINT ASSUMING COMPUTED LINE OF THRUST FOR ($m = 1$) IN ARCH NO. 1.

$$\left(m = 1.3;\ \text{assumed}\ \frac{H_{DL}}{H_{RS}} = 137.3.\right)$$

Point.	y.	$\frac{z'}{c}$.	$\frac{1}{U'}$.	$\frac{1}{\frac{z'}{c}U'}$.	$\frac{1}{\left(\frac{z'}{c}U'\right)^{1.3}}$.	cos a.	$(10 \cos a)^2$.	$\cos^5 a$.	$\cos^6 a$.	$y^{1.3}$	$f\frac{d\phi}{ds}$	Column (5) × Column (7) × Column (9) × Column (10)	Column (11) × y.	$\frac{u'}{c}$	Column (5) × Column (7) × Column (8) × Column (10) × Column (13)
	(1)	(2)	(3)	(4)	(5)	(6)	(7).	(8)	(9)	(10)	(11)		(12)	(13)	(14)
1	−24.60	0.0685	0.186	2.72	3.67	0.725	1.485	0.201	0.145	64.6	51.0	......	1255	3.97	267
2	−18.30	0.0592	0.163	2.75	3.72	0.762	1.500	0.256	0.195	43.8	47.7	......	862	4.60	289
3	−12.68	0.0475	0.137	2.88	3.96	0.800	1.515	0.328	0.262	27.3	43.0	......	545	5.60	301
4	− 7.80	0.0336	0.103	3.08	4.32	0.838	1.530	0.413	0.346	14.50	33.2	......	259	7.90	313
5	− 3.60	0.0175	0.058	3.28	4.68	0.876	1.542	0.518	0.453	5.35	17.5	......	62	14.2	285
6	− 0.07	0.00037	0.002	4.0	6.0	0.912	1.556	0.632	0.576	.....	....	......	....	.....	295
7	+ 2.72	0.0168	0.057	3.42	4.95	0.944	1.558	0.748	0.707	3.65	....	−19.9	54	14.9	312
8	+ 4.82	0.0323	0.101	3.12	4.38	0.970	1.572	0.856	0.831	7.72	....	−44.3	214	8.30	380
9	+ 6.22	0.0441	0.130	2.95	4.07	0.989	1.580	0.945	0.936	10.73	....	−64.5	401	6.23	405
10	+ 6.91	0.0509	0.146	2.88	3.96	0.999	1.59	0.995	0.994	12.35	....	−77.5	545	5.38	417
Σ	....	......	.....	....	....	.....	....	.....	.....	.....	192.5	−206.2	4297	.....	3264

Columns (13), (14), and (15) have been added to compute the change in span length due to rib-shortening. The distances from the axis of the arch to the center of rotation of the section, which are given as $\frac{u'}{c}$ in Column (14), are taken from Fig. 9 for the corresponding values of $\frac{z'}{c}$ computed in Column (11). A check is thus obtained on the rib-shortening thrust by comparing the lengthening of the arch due to angle changes (the summation of Column (7)) and the shortening of the arch due to rib-shortening (the summation of Column (15)).

TABLE 7.—COMPUTATIONS FOR THE LOCATION OF THE NEUTRAL POINT, AND CO-ORDINATES OF THE ARCH POINTS FOR ARCH NO. 2.

Point.	Δx.	x.	$\frac{dy}{dx}$.	$\cos a$.	$\cos^5 a$.	$y_o = \frac{x^2}{2k}$.	$y_o \cos^5 a$.	y.
1	7.0	66.3	0.475	0.903	0.600	15.80	9.48	—10.82
2	7.0	59.5	0.425	0.920	0.658	12.65	8.32	— 7.67
3	7.0	52.5	0.375	0.936	0.719	9.84	7.07	— 4.86
4	7.0	45.5	0.325	0.951	0.780	7.40	5.77	— 2.42
5	7.0	38.5	0.275	0.964	0.830	5.30	4.40	— 0.32
6	7.0	31.5	0.225	0.976	0.887	3.54	3.14	1.44
7	7.0	24.5	0.175	0.985	0.927	2.14	1.98	2.84
8	7.0	17.5	0.125	0.992	0.961	1.09	1.05	3.89
9	7.0	10.5	0.075	0.997	0.983	0.39	0.38	4.59
10	7.0	3.5	0.025	0.9997	0.998	0.04	0.04	4.94
Σ	...	....	.....	......	8.343	.....	41.63	....

Computations for the relative displacements at the ends of the imaginary cantilevers connected to the arch springings and terminating at the neutral point, are given in Table 9 for the equilibrium polygon satisfying the ordinary theory of elasticity. The elasticity of the arch material is considered to satisfy the exponential stress-strain equation when $m = 1.3$. In this, as in Arch No. 1, the sum of the differential angle changes is practically equal to zero, although the lengthening due to angle changes (the summation of Column (12)) is greater than the shortening due to deformations along the axis of the rib (the summation of Column (14)).

In Table 10, calculations for angle changes and rib-shortening deformations were made considering that $\frac{H_{DL}}{H_{RS}} = 81.8$. It will be noticed that, for this ratio of the horizontal thrusts due to dead load and rib-shortening, respectively, the sum of the angle changes is practically equal to zero, and the change in span of the arch due to rib-shortening (summation of Column (15)) and the change in span due to angle changes (summation of Column (13)) are nearly equal in magnitude. Thus, since the line of thrust satisfies the conditions of statics and the geometric requirements imposed, it may be concluded that the line of thrust on which the computations in Table 10 are based, is correct.

TABLE 8.—COMPUTATION OF RATIO OF HORIZONTAL THRUSTS FOR RIB-SHORTENING AND UNIFORMLY DISTRIBUTED LOAD AND THE CORRESPONDING ECCENTRICITY OF THE LINE OF THRUST FOR ARCH NO. 2.

$(m = 1;\ 2\,c_0 = d_0 = 2.)$

Point.	Δx. (1)	y. (2)	$\cos \alpha$. (3)	$\cos^3 \alpha$. (4)	$\cos^5 \alpha$. (5)	y^2. (6)	$y^2 \cos^5 \alpha$. (7)	$\cos^2 \alpha$. (8)	$y \cos^3 \alpha$. (9)	$\frac{H_{DL}}{H_{RS}} - \cos^2 \alpha$. (10)	$\frac{z'}{c} = \frac{\text{Column (9)}}{\text{Column (10)}}$. (11)	$z' = \frac{\text{Column (11)}}{\text{Column (8)}}$. (12)	$\cos^4 \alpha$. (13)	$\frac{u'}{c}$. (14)	Column (2) × Column (13) × Column (14). (15)
1	7	−10.82	0.903	0.736	0.600	117.1	70.26	0.815	−7.96	62.3	0.1278	0.1569	0.664	2.62	18.8
2	7	− 7.67	0.920	0.779	0.658	58.8	38.70	0.846	−5.97	62.3	0.0958	0.1132	0.716	3.50	19.1
3	7	− 4.86	0.936	0.820	0.719	23.6	16.98	0.876	−3.99	62.2	0.0641	0.0732	0.769	5.18	19.3
4	7	− 2.42	0.951	0.880	0.780	5.85	4.56	0.905	−2.08	62.2	0.0334	0.0369	0.819	9.90	19.6
5	7	− 0.32	0.964	0.895	0.830	0.10	0.08	0.928	−0.29	62.2	0.0047	0.0051	0.862	.....	19.9*
6	7	+ 1.44	0.976	0.929	0.887	2.06	1.83	0.952	1.34	62.1	0.0216	0.0227	0.905	15.50	20.2
7	7	2.84	0.985	0.956	0.927	8.07	7.48	0.970	2.72	62.1	0.0438	0.0452	0.941	7.65	20.4
8	7	3.89	0.992	0.976	0.961	15.13	14.54	0.988	3.80	62.1	0.0612	0.0621	0.970	5.43	20.5
9	7	4.59	0.997	0.991	0.983	21.1	20.75	0.994	4.55	62.1	0.0733	0.0737	0.988	4.52	20.5
10	7	4.94	0.9997	0.999	0.998	24.4	24.35	0.999	4.94	62.1	0.0795	0.0796	0.999	4.17	20.6
Σ	.	....	9.624	8.941	.....	......	199.53	.....	......	....	......	......	.....	.....	198.9

* Interpolated.

TABLE 9.—COMPUTATIONS FOR RELATIVE DISPLACEMENT AT ENDS OF CANTILEVERS TERMINATING AT NEUTRAL POINT FOR EQUILIBRIUM POLYGON SATISFYING ELASTIC THEORY FOR ARCH NO. 2.

$$\left(m = 1.3;\ \text{assumed}\ \frac{H_{DL}}{H_{RS}} = 63.1.\right)$$

Point.	y.	$\frac{z'}{c}$.	$\frac{1}{U'}$.	$\frac{1}{\frac{z'}{c}U'}$.	$\frac{1}{\left(\frac{z'}{c}U'\right)^{1.3}}$.	cos α.	$(10 \cos \alpha)^{0.2}$.	$\cos^5 \alpha$.	$\cos^6 \alpha$.	$y^{1.3}$.	$\frac{d\phi}{ds}$ = Column (5) × Column (7) × Column (9) × Column (10).		$\frac{d\phi}{ds} \times y$.	$\frac{u'}{c}$.	Column (5) × Column (7) × Column (8) × Column (10) × Column (13).
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)		(12)	(13)	(14)
1	−10.82	0.1278	0.294	2.30	2.95	0.903	1.558	0.600	0.542	22.20	−55.1	....	595	2.03	124
2	− 7.67	0.0958	0.240	2.50	3.30	0.920	1.560	0.658	0.606	14.20	−44.3	....	340	2.63	126
3	− 4.86	0.0641	0.176	2.74	3.72	0.936	1.565	0.719	0.671	7.80	−30.5	....	148	4.04	132
4	− 2.42	0.0334	0.103	3.08	4.32	0.951	1.570	0.780	0.741	3.15	−15.9	....	39	7.85	131
5	− 0.32	0.0047	0.017	3.7	5.5	0.964	1.57	0.830	0.800	0.23	− 1.6	....	1	....	127*
6	1.44	0.0216	0.071	3.29	4.69	0.976	1.57	0.887	0.865	1.61	....	10.2	15	11.7	123
7	2.84	0.0438	0.127	2.90	3.99	0.985	1.58	0.937	0.923	3.88	....	22.7	64	6.05	139
8	3.89	0.0612	0.169	2.76	3.74	0.992	1.58	0.961	0.953	5.85	....	33.0	130	4.21	140
9	4 59	0.0733	0.195	2.66	3.56	0.997	1.582	0.983	0.980	7.25	....	40.0	184	3.50	141
10	4.94	0.0795	0.208	2.62	3.50	0.9997	1.586	0.998	0.998	7.97	....	44 0	218	3.18	141
Σ	....	......	.....	....	....	......	.....	.....	.....	....	−147.4	149.9	1 734	....	1 324

* Interpolated.

TABLE 10.—COMPUTATION FOR ECCENTRICITY, $\frac{z'}{c}$, DUE TO RIB-SHORTENING WITH UNIFORMLY DISTRIBUTED LOAD ON ARCH NO. 2.

$$\left(m = 1.3; \frac{H_{DL}}{H_{RS}} = 81.8.\right)$$

Point.	y. (1)	$\cos\alpha$. (2)	$y\cos^3\alpha$. (3)	$\frac{z'}{c}$.* (4)	$\frac{1}{U'}$. (5)	$\frac{1}{\left(\frac{z'}{c}U'\right)}$. (6)	$\frac{1}{\left(\frac{z'}{c}U'\right)^{1.3}}$. (7)	$(10\cos\alpha)^{0.2}$. (8)	$\cos^5\alpha$. (9)	$\cos^6\alpha$. (10)	$y^{1.3}$. (11)	$f\frac{d\phi}{ds}$ = Column (7) × Column (8) × Column (10) × Column (11). (12)		$f\frac{d\phi}{ds}y$. (13)	$\frac{u'}{c}$ (14)	Column (7) × Column (8) × Column (9) × Column (11) × Column (14). (15)	$\cos^2\alpha$. (16)	$z' = \frac{\text{Column (4)}}{\text{Column (16)}}$ (17)
1	−10.82	0.903	−7.96	−0.0985	0.243	2.47	3.23	1.55	0.600	0.542	22.20	−60.2	.....	651	2.54	169	0.815	−0.1210
2	− 7.67	0.920	−5.97	−0.0738	0.195	2.64	3.52	1.56	0.658	0.606	14.20	−47.3	.....	363	3.45	177	0.846	−0.0872
3	− 4.86	0.936	−3.99	−0.0493	0.143	2.90	3.99	1.56	0.719	0.671	7.80	−32.6	.....	158	5.40	188	0.876	−0.0562
4	− 2.42	0.951	−2.08	−0.0256	0.082	3.19	4.52	1.57	0.780	0.741	3.15	−16.5	.....	40	9.75	170	0.905	−0.0284
5	− 0.32	0.964	−0.288	−0.0035	0.014	4.0	6.0	1.57	0.830	0.800	0.23	− 1.7	.....	1	....	165†	0.929	−0.0038
6	1.44	0.976	1.34	+0.0166	0.057	3.43	4.95	1.57	0.887	0.865	1.61	......	10.8	16	14.90	165	0.952	+0.0174
7	2.84	0.985	2.72	0.0337	0.103	3.06	4.27	1.58	0.927	0.923	3.88	......	24.2	69	7.80	189	0.970	0.0348
8	3.89	0.992	3.80	0.0470	0.137	2.92	4.02	1.58	0.961	0.953	5.85	......	35.4	138	5.69	202	0.986	0.0477
9	4.59	0.997	4.55	0.0563	0.158	2.79	3.80	1.58	0.983	0.980	7.25	......	42.7	196	4.70	202	0.994	0.0567
10	4.94	0.9997	4.94	0.0611	0.170	2.78	3.78	1.58	0.998	0.998	7.97	......	47.6	235	4.23	202	0.999	0.0611
Σ	....	.....	....	....	.....	....	....	....	.....	.....	.....	−158.3	160.7	1867	.....	1829	.....	....

---

* Since in Equation (87) the value, $\cos^2\alpha$, causes only an inappreciable variation in $\frac{z'}{c}$, $\frac{H_{DL}}{H_{RS}} - \cos^2\alpha$ was assumed to be constant and equal to 80.9.

† Interpolated.

Both Tables 8 and 10 indicate the eccentricity of the line of thrust at various points along the rib for Arch No. 2 under uniformly distributed vertical loading. For the case in which strain varies more rapidly than stress, that is, when $m = 1.3$ (Table 10), the line of thrust lies closer to the arch axis than when stress is considered proportional to strain, that is, when $m = 1.0$ (Table 8).

Studies similar to these have been made for non-uniform loading over an arch. In these investigations, which are not here included, it was found that, neglecting rib-shortening, the lines of thrust were practically identical for different values of m. Rib-shortening influences become greater for low than for high values of m.

A STUDY OF THE EFFECT OF SHRINKAGE

Arch No. 2 will be considered to be loaded with a uniformly distributed vertical load, and subject to plastic, elastic, and shrinkage deformations. The shrinkage will be assumed to be a definite function of the time, and the plastic flow will be considered to be a function of the time and stress intensity. In order to make the problem as simple as practicable it will be considered that no shrinkage causing internal stresses occurs until the centering supporting the newly constructed arch is removed. At that moment, all deformations are due to elastic distortions, and plastic and shrinkage deformations begin simultaneously. The problem, then, is to find the location of the line of thrust in the arch at the end of any time interval after striking the centering.

Consider the properties of the material to be defined by the exponential equations with the following parameters: $m = 1.3$; $p = 1.3$; $k = 183 \times 10^{-9}$; $K = 17 \times 10^{-9}$; and $q = 0.40$.

The eccentricity of the line of thrust at the crown when the structure is in equilibrium elastically before any plastic or shrinkage deformations have taken place, has been found to be $0.0611 \times 12 = 0.733$ in. With the stated conditions, since $m = p$ and neglecting shrinkage, there could be no movement of the line of thrust due to plastic flow.

Assume that the shrinkage is equal to,

$$\gamma = \eta \, t^{q} \dots\dots\dots\dots\dots\dots\dots\dots\dots (103)$$

in which

$$\eta = 145 \times 10^{-7}$$

This would result in a unit shrinkage of about 0.0002 in two years.

The equations, stipulating, respectively, that the relative horizontal displacement of the springings and the total angle change in the arch rib shall be zero, may be written for Arch No. 2, as follows:

$$\frac{10^{-9.2}\,(183 + 17\,t^{0.4})}{(144)^{1.3}} \left[\sum \left(\frac{M}{\frac{z'}{c}\,U'} \right)^{1.3} (\cos\alpha)^{6}\,(10\cos\alpha)^{0.2}\,y_{s}\,\Delta x \right.$$

$$\left. + \sum \left(\frac{M}{\frac{z'}{c}\,U'} \right)^{1.3} (\cos\alpha)^{5}\,(10\cos\alpha)^{0.2}\,\frac{v'}{c}\,\Delta x \right] - 140 \times 145 \times 10^{-7}\,t^{0.4} = 0. (104)$$

$$\frac{10^{-9.2}\,(183 + 17\,t^{0.4})}{(144)^{1.3}}\left[\sum\left(\frac{M}{\frac{z'}{c}\,U'}\right)^{1.3}(\cos\alpha)^{6}\,(10\cos\alpha)^{0.2}\,\Delta x\right] = 0 \quad\text{..(105)}$$

Before any effects of shrinkage or plastic flow have taken place, the ratio of the dead load horizontal thrust to the rib-shortening horizontal thrust will be 81.8. This calculation has been indicated in Table 10. Since shrinkage takes place, one might well presuppose that the rib-shortening thrust will become greater. It will be considered, therefore, that the ratio, $\frac{H_{DL}}{H_{RS}}$, is equal to 71.8. Computations for the summations are shown in Table 11. By substituting these summations in Equation (105), and collecting the terms, the following numerical relation is obtained:

$$t = \left[-\frac{698}{64.7 - \dfrac{2\,090\,000}{H_{RS}^{1.3}}}\right]^{2.5} \quad\text{.........................(106)}$$

Assuming that the uniformly distributed load causes an average thrust of 350 lb. per sq. in. at the crown,

$$H_{RS}^{1.3} = \left(350 \times 12 \times 24 \times \frac{1}{71.8}\right)^{1.3} = (1\,405)^{1.3} = 12\,400$$

and,

$$t = \left[-\frac{698}{64.7 - \dfrac{2\,090\,000}{12\,400}}\right]^{2.5} = 118 \text{ days}$$

The elastic change in length of span resulting from this rib-shortening thrust would be,

$$\Delta l = \frac{183}{(144)^{1.3}} \times 10^{-9}\,(1\,841 - 1\,477)\,2 \times 7 \times 10^{-0.2}\,H_{RS}^{1.3} = 674 \times 10^{-9}\,H_{RS}^{1.3}$$

For a load producing a crown thrust of 350 lb. per sq. in., the change in span length would be, $\Delta l = 674 \times 10^{-9} \times 12\,400 = 0.00836$ ft., or 0.100 in., which is extremely small.

To take shrinkage into account, it has been considered that the crown eccentricity of the thrust line would be a total of $0.0696 \times 12 = 0.836$ in., or $0.836 - 0.733 = 0.103$ in. greater than the normal eccentricity when the structure is in elastic equilibrium. As a result, of course, when the time interval, t, is zero, a slight horizontal displacement of the abutments ($\Delta l =$ 0.1 in.) is assumed. A period of 118 days, then, is the time which it would take for this horizontal displacement to become zero, considering that both plastic flow and shrinkage follow the assumed laws and that the average crown thrust is 350 lb. per sq. in. Thus, the eccentricity of the thrust line would become somewhat greater.

Consider, now, the case in which the denominator in the second member of Equation (106) is zero. That is,

$$64.7 - \frac{2\,090\,000}{H_{RS}^{1.3}} = 0$$

TABLE 11.—COMPUTATIONS FOR RELATIVE MOVEMENT OF SPRINGINGS OF ARCH No. 2, WHEN UNIFORMLY LOADED AND $m = 1.3$; $\frac{H_{DL}}{H_{RS}} = 71.8$

Point.	y.	$\frac{z'}{c}$.*	$\frac{1}{U'}$.	$\frac{1}{\left(\frac{z'}{c}U'\right)}$.	$\frac{1}{\left(\frac{z'}{c}U'\right)^{1.3}}$.	$\cos\alpha$.	$(10\cos\alpha)^{0.2}$.	$\cos^5\alpha$.	$\cos^8\alpha$.	$y^{1.3}$.	$f\frac{d\phi}{dx}$ = Column (5) × Column (7) × Column (9) × Column (10).		$\frac{u'}{c}$.	Column (5) × Column (7) × Column (8) × Column (10) × Column (12)	y_s.	$fy_s\frac{d\phi}{dx}$.	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)		(12)	(13)	(14)	(15)	
1	−10.82	0.1125	0.268	2.38	3.08	0.903	1.55	0.600	0.542	22.20	− 57.5	.....	2.26	144	1.70	98.7	.....
2	− 7.67	0.0842	0.218	2.59	3.44	0.920	1.56	0.658	0.606	14.20	− 46.3	.....	2.98	150	4.85	224.5	.....
3	− 4.86	0.0563	0.158	2.80	3.81	0.936	1.56	0.719	0.671	7.80	− 31.1	.....	4.67	156	7.66	238.2	.....
4	− 2.42	0.0294	0.094	3.19	4.51	0.951	1.57	0.780	0.741	3.15	− 16.4	.....	8.70	152	10.10	165.6	.....
5	− 0.32	0.0041	0.016	3.9	5.7	0.964	1.57	0.830	0.800	0.23	− 1.6	.....	....	148†	12.20	19.5	.....
6	1.44	0.0189	0.063	3.33	4.78	0.976	1.57	0.887	0.865	1.61	....	10.4	12.90	143	13.96	.....	145.2
7	2.84	0.0384	0.116	3.03	4.22	0.985	1.58	0.927	0.923	3.88	....	23.9	6.85	165	15.36	.....	367.0
8	3.89	0.0536	0.152	2.84	3.88	0.992	1.58	0.961	0.953	5.85	....	34.2	4.98	173	16.	.....	562.0
9	4.59	0.0642	0.176	2.74	3.72	0.997	1.58	0.983	0.980	7.25	....	41.7	4.08	173	17.41	.....	713.5
10	4.94	0.0696	0.188	2.70	3.63	0.9997	1.58	0.998	0.998	7.97	....	45.8	3.78	173	17.46	.....	800.0
Σ	......	......	.....	....	....	......	....	.....	.....	....	−152.9	156.0	....	1577	.....	746.5	2587.7

*Assumed that $\frac{H_{DL}}{H_{RS}} - \cos^2\alpha = 70.9$ since the greatest variation from this value is 0.1 and thereby causes an inappreciable effect.

† Interpolated.

or,

$$H_{RS} = \left(\frac{2\ 090\ 000}{64.7}\right)^{\frac{1}{1.3}} = 2\ 920 \text{ lb.}$$

Then, the average dead load thrust at the crown will be,

$$\frac{H_{DL}}{12 \times 24} = \frac{2\ 920 \times 71.8}{12 \times 24} = 728 \text{ lb. per sq. in.}$$

Thus, if the average thrust at the crown were 728 lb. per sq. in., or a total horizontal thrust of 728 × 24 × 12 = 210 000 lb., the rate of change of span length due to plastic deformation would be just equal and opposite to that due to shrinkage. As a result, t becomes infinite.

If the properties of the concrete approximate those assumed for the analysis, the foregoing considerations make it difficult to conceive how shrinkage of the concrete can materially affect the stresses in the arch.

Deflection of Crown of Arches

General Equations.—Fig. 28 indicates a loaded arch, the line of thrust being indicated by a full line.

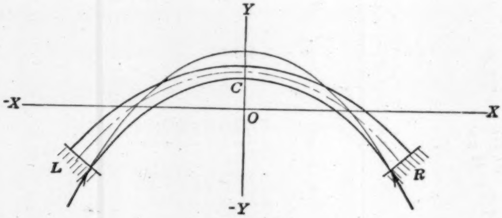


Fig. 28.

If the springing, L, does not rotate, the vertical deflection of the crown, C, must be,

$$\delta_c = \int_C^L \frac{d\,\phi}{d\,s}\,\frac{x\,d\,x}{\cos\alpha} + \int_C^L \frac{d\,\xi}{d\,s}\tan\alpha\,d\,x \ldots\ldots\ldots\ldots (107)$$

regardless of whether the deformations are elastic, plastic, or a combination of both. Replacing the definite integrals of Equation (107) by finite summations, and considering only elastic deformations, the deformations following the relations,

$$\frac{d\,\phi}{d\,s} = k\left(\frac{M}{z'\,U}\right)^m, \text{ and } \frac{d\,\xi}{d\,s} = k\,u'\left(\frac{M}{z'\,U}\right)^m$$

there results the following equation for the vertical deflection of the crown of the arch:

$$\delta_c = k\sum_C^L \left(\frac{M}{z'\,U}\right)^m \frac{x\,\Delta\,x}{\cos\,\alpha} + k\sum_C^L u'\left(\frac{M}{z'\,U}\right)^m \tan\,\alpha\,\Delta\,x \ldots\ldots (108)$$

If Hooke's law is considered to hold, Equation (108) may be rewritten in the form,

$$\delta_c = \sum_C^L \frac{M\,x\,\Delta\,x}{E\,I\cos\,\alpha} + \sum_C^L \frac{T\tan\,\alpha\,\Delta\,x}{E\,A} \ldots\ldots\ldots\ldots (109)$$

In case both elastic and plastic deformations are considered, Equation (107) could be written as,

$$\delta_c = k \sum_{C}^{L} \left(\frac{M}{z' U}\right)^m \frac{x \Delta x}{\cos \alpha} + K t^q \sum_{C}^{L} \left(\frac{M}{z' V}\right)^p \frac{x \Delta x}{\cos \alpha}$$

$$+ k \sum_{C}^{L} u' \left(\frac{M}{z' U}\right)^m \tan \alpha \, \Delta x + K t^q \sum_{C}^{L} u' \left(\frac{M}{z' V}\right)^p \tan \alpha \, \Delta x \ldots (110)$$

If $n = p = m$, then $W = V = U$, and Equation (110) simplifies to,

$$\delta_c = (k + K t^q) \left[\sum_{C}^{L} \left(\frac{M}{z' W}\right)^n \frac{x \Delta x}{\cos \alpha} + \sum_{C}^{L} \left(\frac{M}{z' W}\right)^n w' \tan \alpha \, \Delta x \right] . (111)$$

Vertical Deflection of Crown of Arch No. 2 When Subjected to a Uniformly Distributed Vertical Load.—Arch No. 2 is considered to be loaded with a uniform load of w lb. per ft. of length. Tabular computations, not here included, based on the foregoing equations, were carried out to study the crown deflection due to rib-shortening effects when m is equal to 1 and 1.3, respectively.

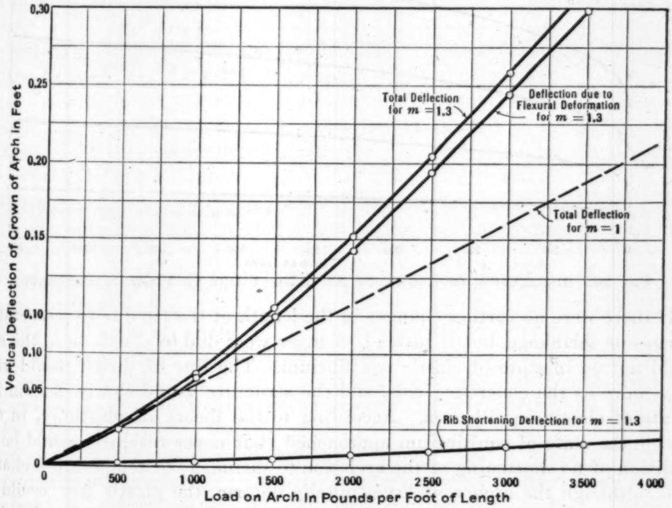


FIG. 29.—VERTICAL DEFLECTION OF CROWN OF ARCH NO. 2.

In order to have a basis for comparison of the deflection of the crown of an arch composed of a concrete following Hooke's law with one in which the exponential equation governs the relation of stress to strain, the parameters, E and k, have been chosen so that the deflection will be the same for both arches when the average stress at the crown section is 300 lb. per sq. in.

In Fig. 29 are shown the crown deflections of the arch for various magnitudes of the uniformly distributed load. For the case of a variable modulus of elasticity, several curves have been drawn; only the total deflection has been plotted for the arch in which material of construction is assumed to have a constant modulus of elasticity.

Crown Deflection, Considering Plastic Flow.—As a simple example of the anticipated effect of plastic flow on the crown deflection of an arch, Arch No. 1 is considered to be subjected to a uniformly distributed load and to be undergoing rib-shortening as well as temperature change. The following numerical values are used: $m = p = 1.3$; $k = 613 \times 10^{-10}$; $K = 56.7 \times 10^{-10}$; and $q = 40$.

It is considered that the ratio, $\dfrac{H_{DL}}{H_{RS}}$, is equal to 25. In this case, the rib-shortening force, H_{RS}, is due to all causes, including temperature change, shrinkage, and rib-shortening. The location of the resulting line of thrust is indicated in Table 2, together with computations for angle changes in the arch rib.

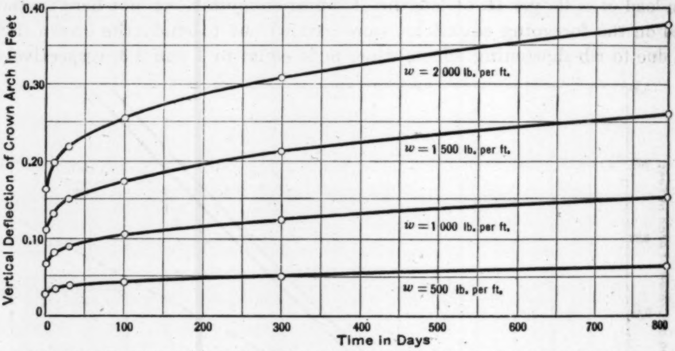


FIG. 30.—DEFLECTION OF CROWN OF ARCH NO. 1 DUE TO FLOW OF CONCRETE.

If there were no further changes in the length of the rib due to temperature changes or shrinkage, but if instead, changes were due to plastic flow, the arch would not be in state of elastic equilibrium. The line of thrust would gradually move as the concrete yields, and the structure would slowly deform into a state of elastic equilibrium. According to the theory of plasticity, in case $m = p$, the state of equilibrium approached (but never reached) would be the condition of no shortening of the arch due to shrinkage or temperature change. Thus, although the arch would continually deform, the plastic flow would act in such a way as to bring the structure back to its original state of equilibrium.

However, for simplification, and as a hypothetical case, the structure is supposedly held in elastic equilibrium by assuming that the temperature drops at such a rate that the tendency of the plastic flow to bring the structure to its original state of equilibrium is just offset by the reverse action caused by temperature change. As a result, the structure changes in shape although the relative location of the line of thrust and the stress distribution remains unchanged. In Fig. 30 are curves based on computations for various uniform load distributions over the arch.

CONCLUSION

Inasmuch as the investigation presented in this paper is based on an impartial interpretation of physical facts regarding the behavior of concrete under sustained load, the writer feels justified in pointing out a few conclusions which appear unquestionable as a result of his study:

First.—It is incorrect to assume that in general plastic flow in statically indeterminate concrete structures causes a redistribution of bending moments tending to relieve the overstressed parts of the structure.

Second.—If there is an external disturbance encountered by a statically indeterminate concrete structure, such as the settlement of a support, the plastic flow causes a gradual redistribution of stresses such that the structure approaches (but never reaches) its original state of equilibrium (no settlement of support). When no external disturbances occur, plastic flow will cause progressive changes in the deflections and displacements of points on the structure, but will cause little or no change in the moment distribution. In this connection, temperature changes as well as rib-shortening and shrinkage in arches are included as external disturbances.

Third.—Provided disturbances due to reinforcing steel are not important, it is evident that striking the centering of a fixed arch at an early date after pouring the concrete has many advantages. In spite of the fact that the total rib-shortening resulting will be greater, the stresses occasioned thereby and in consequence of the immediate movement of the abutments will be less. The greatest effects on the stress distribution due to rib-shortening and movement of abutments will occur when the centering is allowed to support the arch far beyond the time required, although for this case the total rib-shortening will probably be a minimum.

Fourth.—As far as all experimental data on concrete examined by the writer are concerned, plastic flow would cause the line of thrust in an arch to approach the center axis more closely. However, it is conceivable that plastic flow might cause the line of thrust to be pulled away from the axis; this will occur when $m > p$.

Fifth.—In making analyses of the rib-shortening stresses in an arch it is of vital importance to distinguish between shrinkage, temperature change, and elastic deformation on the one hand, and plastic deformation on the other.

Sixth.—The fact that the ratio of stress to elastic strain does not follow Hooke's law for concrete usually causes but little difference in moment distribution from that computed by the ordinary elastic theory. For rib-shortening and temperature changes the difference may be greatly on the side of safety.

Seventh.—Stresses computed by the ordinary elastic theory for temperature changes are likely to be greatly in error, but on the side of safety for high temperature variations.

Eighth.—From the analyses of the crown deflections of an arch it appears well to sound a warning against misinterpretation of test data. The writer believes, especially for newly constructed arches, that, although the crown deflections measured in stressed arches may check computations made by the ordinary theory of elasticity in many cases, the location of the line of thrust may be very different from that indicated by working back from the measured deflection by the usual theory.

The new theories presented in this paper are not intended to replace the ordinary method of analyses used heretofore, but are developed with the idea of supplementing the older theory of elasticity. For important statically indeterminate structures, such as long-span, fixed concrete arch bridges, an analysis seems incomplete without an investigation by more accurate methods than is provided by the usual theory in which stress is assumed to be proportional to strain and deformation due to flow is neglected. For smaller structures the usual procedure for stress analysis is probably satisfactory. The proposed methods serve further as tools for engineers in research and help to develop more accurate conceptions regarding the behavior of concrete structures.

Much more experimental work, especially of a quantitative nature, is desirable and necessary in order to establish definitely the laws of plastic flow for concrete of various mixes and ages.

DISCUSSION

B. F. Jakobsen,[22] M. Am. Soc. C. E. (by letter).—The author presents a theory of bending in which the stress-strain relation of Equation (9), $\epsilon = k\,\sigma^m$, is substituted for Hooke's law, for which $m = 1$. As presented by the author, the constants, k and m, are the same for tension as for compression.

The advantage of Hooke's law is its simplicity, as may be seen by comparing the ordinary formulas with those presented by the author; another advantage is that if it errs, as it does for high stresses, it errs on the side of safety. An analysis by the writer[23] of the test on the arch at Elevation 30 of the Stevenson Creek Dam shows that Cain's formulas, based on a constant modulus of elasticity, give stresses which agree well with the measured strains. In this case the stresses are fairly small and they are compressive throughout. An attempt to deal with the arch at Elevation 40 failed,[24] probably because the Cain formulas give considerable tension, and either cracks were produced or a shifting of the neutral axis took place, and possibly both phenomena occurred.

It appears to be a moot question whether or not Hooke's law applies for stresses well below the breaking stress, although scientific men have been occupied with a study of this phenomena for two centuries. Bulffinger proposed the exponential law in 1729 and Hodgkinson considered it in 1822. The parabolic law and recently the hyperbolic law have also been considered.[25] Bach states[26] that for compression of concrete, which has not been previously stressed, the deformations increase faster than the stresses. He gives various values for the coefficients, k and m. Stanton Walker, M. Am. Soc. C. E., comes to the same conclusion[27] for concrete in compression. The late J. B. Johnson, M. Am. Soc. C. E.,[28] showed that for tension Hooke's law applies in general until near the breaking strength and states that the modulus of elasticity for stone concrete in tension is only 0.7 as great as in compression. Hool and Johnson give stress-strain curves for concrete in compression,[29] which show that Hooke's law applies for stone concrete up to a stress of more than 2 000 lb. per sq. in. at the age of 1 year. Messrs. Taylor and Thompson state that for compression the deformation curve is approximately a parabola, but that the modulus of elasticity is nearly constant for about one-third the ultimate strength, that is for all normal working stresses.[30] The modulus for tension, they state, is probably the same as for compression, though but few tests have been made. Tests by Mörsch indicate that for concrete the deformations increase faster than the stresses, both for tension and for compression, and that the modulus for tension is slightly larger than for compression.[31]

---

[22] Cons. Engr., Los Angeles, Calif.

[23] *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1241.

[24] *Loc. cit.*, p. 1247.

[25] Bach, "Elasticität und Festigkeit," Ninth Edition (1924), p. 109, footnote.

[26] *Loc. cit.*, p. 84.

[27] *Bulletin No. 5*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill., 1920.

[28] "The Materials of Construction," Fourth Edition (1912), Fig. 549-a, p. 604-a.

[29] Concrete Engineers' Handbook (1918), p. 250, quoting tests made at the Structural Materials Laboratory at St. Louis, Mo.; U. S. Bureau of Standards, *Technical Paper No. 2*.

[30] "Concrete, Plain and Reinforced," Second Edition (1912), p. 404.

Tests have been carried out recently in the Research Laboratory of the Portland Cement Association covering the modulus of elasticity of concrete in both tension and compression, using the most accurate technique and apparatus. As a result of these tests F. R. McMillan, M. Am. Soc. C. E., has stated to the writer under date of January 11, 1930, that he does not accept the exponential law, as Hooke's law applies up to normal working stresses and generally considerably beyond these; also that the modulus in tension is substantially equal to that in compression.[32]

These conflicting opinions are due no doubt to the fact that so many circumstances influence the result. Some of these are the permanent set, previous loading, how the load is applied, whether in steps or continuously, repeated loading and unloading in order to obtain the final deformation for a certain load, the time element, internal stresses due to temperature, shrinkage, etc. In view of these facts it appears to the writer that the usual assumption, that Hooke's law holds, is to be preferred for the proposed exponential law for stresses which do not exceed the normal working stresses.

It is well known that Hooke's law does not hold for stresses approaching the breaking value and that this must materially influence the stress distribution in concrete arches the writer has pointed out elsewhere.[33] Bach has shown that for cast iron the modulus of rupture in bending is about 1.64 times the tensile strength and he explains that fact by showing that the modulus of elasticity is not constant and is not the same for tension as for compression.[34] This difference brings about a shifting of the neutral axis toward the compression side, so that, at failure, the compression at the extreme fiber is 73% greater than the maximum tensile stress. Recent tests on unreinforced concrete beams show that the modulus of rupture in bending is about 1.65 to 2 times as great as the tensile strength.[35] The exponential law using the same constants, k and m, for tension as for compression, as the author does, cannot explain this great discrepancy between the modulus of rupture in bending and the tensile strength, as the following calculation shows. The bending moment derived from the exponential law, Equation (9), is given by Bach[36] as,

$$M_e = \frac{2\,m}{2\,m+1}\,\sigma\,b\,\frac{h^2}{4} \qquad \text{.......................(112)}$$

in which, σ is the maximum fiber stress, which occurs at the surfaces, the same for tension as for compression, since k and m are the same for both kinds of stresses; b is the width; h, the height; and m, the exponent for σ in Equation (9). For Hooke's law, $m = 1$ and,

$$M_h = \frac{b\,h^2}{6}\,\sigma \qquad \text{.......................(113)}$$

---

[31] "Taschenbuch für Bauingenieure," by Max Foerster (1920), pp. 241–242.

[32] See, also, "Relationship Between Strength and Elasticity of Concrete in Tension and in Compression," by James W. Johnson, *Bulletin 90*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa, May, 1928.

[33] *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 507, and Fig. 19, p. 508.

[34] "Elasticität und Festigkeit," Ninth Edition (1924), p. 298.

[35] Report of the Director of Research, Portland Cement Assoc., November, 1928, p. 170, and Table 9, p. 195

[36] "Elasticität und Festigkeit," Ninth Edition (1924), p. 276.

The ratio of M_e to M_h is,

$$R = \frac{6}{4 + \frac{2}{m}} \qquad (114)$$

If $m = 1.3$, $R = 1.08$, or the bending moment is only 8% greater than that found by Hooke's law. For $m = 2$, which is much too large, $R = 1.2$, which is still much too small. (Mr. Walker[37] gives $m = 1.11$ as an average and Bach[38] gives values of m ranging from 1.145 to 1.207.) It is evident that the exponential law, with the same constants for tension as for compression, cannot explain the discrepancy between the modulus of rupture and tensile strength. When k and m are different for tension and for compression, as is the case with the cast iron investigated by Bach, to which reference has been made, the exponential law gives about 89% of the actual bending moment at failure, while Hooke's law gives only about 61%, assuming, in both cases, that the maximum bending stress equals the tensile strength.

The attempt to deal with the time effect, or the so-called plastic deformation, and its influence upon the stress distribution, is interesting. The author assumes that the exponential law holds for the elastic deformation and that another exponential law,

$$\rho = k\, \sigma^p\, t^q \qquad (115)$$

holds for the plastic deformations. From one set of tests by Fuller and More, the author finds $q = 0.42$, while from another set of tests by Mr. Earl B. Smith, he finds $q = 0.18$; these latter tests appear to have been started when the concrete was only two weeks old, so that during the test the modulus of elasticity, strength, etc., must have been changing rapidly, making the results problematical.

The separation of elastic and plastic deformations seems an artificial procedure. Tests show the combined deformation only, and the stress-strain curve depends upon previous loading, so that the same specimen can not be subjected repeatedly to tests without introducing new factors which influence the result to an unknown extent. Moreover, the plastic deformations disappear in due course of time, and are not in any sense permanent, except when the elastic limit has been exceeded. Bach reports[39] bending tests on wooden beams, which, when stressed to 2 100 lb. per sq. in. for 1.5 min., showed a remanent deflection of 0.10 mm. when the load was removed. After 22 hours of continuous loading, the remanent deflection was 0.34 mm. immediately after the load was removed, and 51 hours later this deflection had decreased to 0.14 mm. Tests on other wooden beams showed that this decrease in the remanent deflection was still going on after 162 hours of no load.

If the stress-strain curve is linear for plastic as well as for elastic deformations, Hooke's law applies, and the time factor affects the deformations but not the load stresses. The effect of the time factor is then to lower the modulus of elasticity and to decrease—very materially—the temperature stresses, since these are not suddenly applied. In order to investigate indeterminate structures properly, one would need to know the deformation as a function of the variable stress and time, and this function would probably be so complicated and its determination so uncertain, judging from the disagreement between authorities regarding the ordinary modulus of elasticity, that engineers would have to rely upon some approximate and simple function, such as Hooke's law, modified by the introduction of a time effect, such as the author suggests.

---

[37] *Bulletin No. 5*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill., 1920, p. 88.

[38] "Elasticität und Festigkeit," p. 84.

[39] *Loc cit.*, p. 111.

C. A. P. Turner,[40] M. Am. Soc. C. E. (by letter).—Experience has led the writer to suppose that plastic flow after the removal of the forms was so small as to be incapable of determination by measurement with an ordinary 20-in. level. Mr. Straub apparently computes the plastic deflection of a 140-ft. arch as 3 in.; and a 300-ft. arch of like relative dimension should be in the neighborhood of 6 in. for a time-flow period of more than 1 000 days.

For the purpose of making some independent observations, the writer ran a line of levels on a series of thirteen concrete arches that had been standing for about five years. Each has a rise of 1 to 4 and a span length, from center to center, of 304 ft. The third, eighth, ninth, and tenth spans differed little from the design grade. In other words, the floor was practically straight regardless of the fact that the levels were taken at a temperature 10 or 15° below freezing. Other spans were not quite as close to grade on the curb line, but, since the curb was not exactly true, the total variation of ½ in. ± was not to be wondered at. The conclusions from these levels are as follows:

1.—That plastic flow capable of being measured definitely by a Y-level has not occurred in these thirteen spans.

2.—That, with a rise of 1:4, temperature changes affect the floor alignment only a negligible amount, although they may result in some slight difference in the elevation of the end abutments and the intermediate spans under wide ranges of temperature.

3.—That because the elevation and alignment of the floor were determined during construction by a 20-in. level, any alleged computation of changes of level which cannot be measured by the same instrument are either imaginary or without practical value.

4.—That the inadmissibility of the proposed theory from the practical standpoint arises from the misleading nature of the laboratory experiments upon which Mr. Straub founded his painstaking algebraic work.

Messrs. E. B. Smith, F. R. McMillan, and A. H. Fuller and C. C. More were mistaken in supposing they had discovered a general phenomenon of plastic flow in concrete as applied to the practical structure. They had merely misinterpreted the law of static equilibrium of internal stress in its various manifestations.

Nearly all materials used for construction purposes are in a state of internal strain. These are of two kinds, residual and elastic. Static permanence of form subsists as long as internal elastic and residual strains are in equilibrium. Any cause upsetting this equilibrium is accompanied by a change in form and external dimensions of the specimen. Thus, a steel bar as received from the mill has been squeezed between the rolls and it leaves the mill in a state of internal strain. Cutting a series of saw kerfs across the top or bottom of the bar as it passed through the rolls will cause a slight curvature along the length of the bar. Similar treatment of bars bent cold in a horseshoe shape discloses the opposite nature of reactions of residual and elastic strain. This enables observers to account for the elevation of the primitive yield point of steel under over-strain and to dismiss fatigue as a phenomenon of residual strain unrelated to elastic stress and the selection of working stresses in rational design.

---

[40] Cons. Engr., Minneapolis, Minn.

Contrary to algebraic theory, range of stress from tension to compression of a moderate amount, so far from injuring sound metal improves its elasticity, ductility, and tenacity as found by Bauschinger. This is brought about by the gradual elimination of residual strain by the work of the applied loads. The increase of the elastic constants is limited to 5% for *E*, nearly 20% for *K*, and no change in the modulus for shear.

Temperature change tends to reduce or eliminate residual strain and likewise also range of stress of moderate intensity. Conversely, range of stress greater than five-eighths the primitive yield point increases or accumulates residual strain raising the yield point value.

The resistance of concrete follows the same statical principles involved in the discussion of steel. The 3 by 24-in. cylinders tested by Smith present equal cross-sectional areas subject to the rapid curing of the surface on the one hand and the slower hardening of the interior on the other. This condition differs widely from the 72 to 100-in. square sections of the arch, in which the surface effects are almost negligible.

Laboratory specimens, therefore, disclose a change of form accompanying the redistribution of internal strain which is practically wanting in larger cross-sections, as evidenced by the survey of the 300-ft. arches previously mentioned.

Briquettes cured in water for two or three years present tensile resistance equal to those cured in the dry if tested immediately; but if kept until half dried the resistance may be reduced 50 per cent. When fully dried, the resistance is restored, thus throwing some light on the time tests of Messrs. Fuller and More.

The residual strain in the McMillan tests arises from a different cause. In the initial crystallization of concrete, expansion takes place, inducing tension in the embedded steel. If the rods are as small as $\frac{3}{8}$ in. or less the tension as measured may be 7 000 to 10 000 lb. per sq. in. This tension in steel induces residual compression in the partly cured concrete envelope, having a reaction in residual strain opposite to elastic compression. Shrinkage accompanies the drying out of the concrete with elimination of steel tension, but the residual strain in the concrete remains until otherwise dissipated. Hence, the tensile element of resistance to flexure of the newly cured beam consists of the joint resistance of the residual strain in the concrete plus some tension in the steel. This residual strain is erroneously treated in textbook theory as direct tension in the concrete. A few repetitions of the load and it is largely dissipated. Temperature changes and time produce a like effect. Hence, the inadmissibility of applying such data to the practical problem of the arch rib.

In "Novum Organum", Bacon insists on clearing the metaphysics of technical science by observation, experiment, and experience. In his classification of the tendencies of the human mind to err he lists, under "Idola Tribus", the proneness of the human mind to suppose in Nature, an order and regularity greater than exists; to support preconceived opinion by affirmative observation, while omitting consideration of negative or opposite phenomena; to generalize from a few observations on the one hand or to give reality to mere abstractions or figments of the mind on the other hand. Under "Idola Theatri", he cites fallacious modes of thinking which from accepted systems of philosophy and erroneous demonstration, leaps to general conclusions.

No line of scientific endeavor needs clearing of the metaphysics of these two classes of error so badly as the common views regarding the elastic resistance of materials. The "Idola Tribus" of this viewpoint appears in the lack of recognition of change of form caused by residual strain under the abstraction of the mind of the research fellow that it is nothing more than a measure of elastic strain.

Readings of deformation under load by an extensometer at the junction of a concrete slab and column are abnormally large. Considering such measurements as elastic strain instead of shrinkage recovery, an observer reached the conclusion that the stress just inside the column was approaching infinity. He stated that, because of this newly discovered fact, in a very short time when the elastic stress finally reached infinity, concrete buildings would fall apart and thus eliminate themselves from the field of competition.

Although shift of load in double bending by the combined action of torsional and bending couples is recognized in the old "Natural Philosophy" of Thomson and Tait, it is unheard of in the standard university textbook of to-day. The wide divergence between bending moment computed from steel stress and applied moment is attributed to the tensile strength[41] of concrete after it is broken in tension.

In line with Bacon's "Idola Theatri", colleges of engineering teach the student the formula of Hooke's law and feel they have accomplished enough in bringing the conception of the young engineer regarding elasticity up to within two centuries of advanced thought in physical science. The application of Hooke's law in flexure is then made by a system of partial differentiation arbitrarily assuming not only that shear strain does not follow Hooke's law, but that it is to be applied only to the horizontal components of the principal stresses, thus fixing the neutral axis at the gravity center of the section instead of at its natural locus along the length of the beam. Upon such half-way application of Hooke's law the student is taught computation of resistance to external shear force near the supports of continuous and simple beams, involving an inconsistency of 200 or 300 per cent.

---

[41] *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1560.

Because longitudinal extension is accompanied by lateral contraction, the student is left in doubt as to whether this lateral deformation involves work. If it does not, is there not something wrong about Hooke's law? And if it does, is not the phenomenon in contravention of the law of conservation of energy? In the absence of instruction in the theory of the thermo-elastic properties of matter these are the confusing doubts which occur to the post-graduate student as observed in the technical press.

Confusion arises from a common but inexact statement that the internal work of deformation equals the external work of the load, when, in fact, this equality is limited to the specific direction of the applied force only. The total internal work is measured by the shift of the intrinsic internal heat energy of vibration from kinetic to potential, or *vice versa,* and is nearly double the external work. Hence, the textbook error in computing temperature stress.

EDWARD GODFREY,[42] M. AM. SOC. C. E. (by letter).—This paper is valuable, not so much because it proposes a new theory for the solution of the arch problem, but because it demonstrates the inapplicability of all elastic theories to concrete arches as well as the practical impossibility of fitting a workable theory to the many sided problem of the concrete arch that will fit accurately into every condition.

For many years the writer has held that the elastic theory is not applicable to concrete or masonry arches.[43] The reason for this is fundamental. In fact, there are many reasons, and they are all fundamental.

The elastic theory pre-supposes a homogeneous and perfectly elastic material in the arch. Concrete is far from homogeneous and far from being perfectly elastic. It would seem superfluous to point out specifically why the elastic theory as applied to the arch required these conditions, but at the expense of appearing pedantic they will be emphasized, for this theory has taken a hold on the profession that seems impossible to shake.

Every engineer knows that to satisfy the elastic theory it is not sufficient that a structure shall show merely what is commonly thought of as elastic properties; that is, it is not sufficient that an arch, for example, can be sprung by a load, and when that load is removed, it will return to approximately the original shape. This, in fact, is all that can be said of a concrete arch. It is about all that was proved by the famous Austrian tests cited by the author. The amount of the spring that "agrees with theory", could be made to agree with many other theories by merely assuming or working out a modulus of elasticity to fit.

The average modulus of elasticity as found by specimen tests, in the Austrian series of tests on blocks, was about 2 700 000 lb. The "practical" value as determined from analysis of a plain concrete arch was 1 430 000 lb.[44] or nearly 100% difference. There were no fixed-ended arches among them. There was a long plain concrete arch and a long Monier arch. These are the tests which the Engineering Profession points to when the elastic theory is questioned as to its applicability to reinforced concrete arches and on the basis of which fixed-ended arches are confidently designed. Because a plain concrete bow between solid abutments deflected in an elastic curve, reinforced concrete arches between settling abutments are designed with fixed ends.

---

[42] Structural Engr., Pittsburgh, Pa.

[43] "Concrete," by Edward Godfrey, M. Am. Soc. C. E., 1908, p. 14; *Transactions*, Am. Soc. C. E., Vol. LXX (1910), pp. 64–65.

[44] *Engineering News*, April 9, 1896.

A uniform load on an arch, and an agreement with a theory in the observed measurements under such load, are far from being sound arguments for the extension of that theory to include unsymmetrical loading. Every engineer knows that a material to be elastic in the meaning of the theory, must be first homogeneous. Some of the Austrian tests were on brick and stone masonry. By no stretch of terms can these be considered homogeneous; hence, agreement with theory means nothing, if that theory is an elastic theory.

Concrete is not homogeneous. The properties of the stones are not the same as those of the mortar. One batch of concrete will differ from another. Interior concrete will differ from exterior concrete. Shrinkage will affect the thin parts more than the thick parts. The unstressed shape of the arch can never be known. When the arch is wet, it will expand, altering the stresses; yet the elastic theory must assume entire homogeneity as well as constancy of condition.

To be elastic the concrete must return to its original shape after being stressed; yet concrete takes a permanent set of an unknown amount and of an amount varying with age and other conditions. It is quite impossible to evaluate these conditions. On this fundamental basis, the elastic theory for arches is eliminated from serious consideration, as Mr. Straub points out.

Fixed-endedness is a vital part of the elastic theory. This means absolutely rigid abutments; it means no horizontal movement and no rotation. Either of these influences varies, materially, the stresses in the arch. Horizontal movement is to be expected in any arch not abutting against vertical rock faces. Rotation, even in an arch of 119 ft. span founded on rock, was reported in a progress report by the Special Committee of the Society on Concrete and Reinforced Concrete Arches.[45]

In still another respect concrete fails to meet the mathematical or engineering criterion of an elastic substance. To be elastic in this sense there must be a constant ratio between stress and strain for all intensities of stress within the allowable limits. This is a fundamental assumption of the elastic theory. Concrete is far from meeting this criterion.

All these fundamental bars to the elastic theory as the solution of the concrete arch problem have been known and emphasized for many years. The elastic theory persists, however, and the most elaborate mathematical computations are carried out on the basis of this theory. There is scarcely any other problem in engineering in which such intricate mathematical processes are utilized. In the face of all the intricacy of the elastic theory and the difficulty of designing an arch by this method it is only to be expected that the advocates should show strong justification for its use and close agreement between theory and fact in the matter of stresses. This, they are unable to do, as repeatedly emphasized by the author.

---

[45] *Proceedings*, Am. Soc. C. E., March, 1924, Society Affairs, p. 292.

It will be urged, as it has been, that these objections to the elastic theory are on the high ground of a demand for perfection, that even steel is not perfectly elastic. It is a far cry from the trifling departures from perfect elasticity and perfect fitting of the unstressed steel arch in the position for which it is designed, to the 100% departures from ideal conditions proved to exist in the case of concrete.

There is a psychological hazard in complex theory. The mere application of the theory, the working out of an intricate problem begets a confidence that is not engendered by an approximate method. Factors of safety are cut to finer limits when stress coefficients are worked out to the fifth decimal place (which is not unheard of), based on erroneous premises. When a method of design is frankly set up as an approximation, the designer will naturally take this into account in choosing a factor of safety. The writer has noted factors of safety given in tenths and hundredths (exceeding unity) where the structure that was "safe" had failed. The psychology of engineering may be of as much importance as its mathematics. After all, engineering is not merely the working out of a mathematical problem, no matter how nicely that problem may develop under the solution applied by higher mathematics.

It is of supreme practical importance that designing be simplified where possible. Comparatively few engineers, perhaps less than 1% of the men engaged in designing, could follow the solution of the concrete arch problem by the elastic theory.

One of the principal considerations in the design of an arch is provision in the cross-section of the arch ring for the thrust due to full uniform live load. This is, in fact, the first consideration in point of time. Symmetrical uniform load is not usually the critical condition of loading, however. If full load on an arch were the critical loading, it would matter little what theory were used. Furthermore, it would make little or no difference whether the arch were considered as fixed-ended or hinged-ended. The simplest "row-of-blocks" method would give the same answer as the most complex elastic theory. It is simply a matter of resolving the forces, applied loads, and end reactions, into the proper equilibrium polygon.

Mr. Straub points out the fact that his theory and the elastic theory are in practical agreement for the condition of uniform live loading. It is when unsymmetrical loading and provision for such loading are taken up that the several theories diverge. The author points out that when the real properties of concrete are theoretically taken into account—the non-elastic properties, so to speak—it is not possible to determine the distribution and intensity of stress on a concrete cross-section, given the direct compression and the bending moment. It is necessary in the solution, the auther shows, to know the point of application of the force producing the bending moment. This not only complicates the solution of the problem on the author's proposed theory, but makes it practically impossible to determine the distribution of live load that will constitute the critical condition for design. The fact is that the problem of determining the critical position of live load for maximum extreme fiber stress introduces such complications into the elastic theory itself that the question is touched upon but lightly in treatises and papers on that subject.

It is significant that arch tests that are used to sustain the elastic theory are not made on critical loading on the arches. It is such testing that would be useful in checking up the theory and furnishing a basis for confidence in it.

The crux of the situation as to concrete arch design centers on the assumption of fixed-endedness, invariably made when the elastic theory is applied. There would be no use whatever for the elastic theory in the design of concrete arches, if the arches were assumed as hinged-ended; and here is another of the anomalies of theoretical engineering. The fixed-endedness of an arch pre-supposes massive, immovable abutments, and yet theoretical treatments of elastic arches gives little or no attention to the proportions of the abutments that are necessary to give the needed rigidity.

It is usually difficult to provide sufficient width, economically, in an abutment to bring the line of thrust within the middle third of the base, assuming the arch to be hinged-ended; and this assumption means a practically fixed line of thrust through the abutment. By the methods of the elastic theory the line of thrust through the abutment for unsymmetrical load, may strike through the upper edge. A large and sometimes prohibitive addition in the mass of the abutment, would be demanded, if this were properly taken into account. The extra concrete required in this case to give the abutment its needed stability far outweighs the saving of a few inches in the thickness of the arch ring which, theoretically, is effected by the methods of the elastic theory and assumed fixed-endedness.

J. H. Griffith,[46] M. Am. Soc. C. E. (by letter).—The writer is impressed with the fundamental character of this paper and by the fact that much thought and careful study have been given to its preparation. Apart from the author's elaboration of his thesis he calls attention from time to time to the disparities existing between the scientific and technologic viewpoints in interpretations of the phenomena of elasticity, plasticity, viscosity, and other properties of matter. Notwithstanding the profound researches upon the subjects by Dr. Eugene Cook Bingham and by Charles Terzaghi, M. Am. Soc. C. E., the author feels the questions at issue are mooted ones. The writer would like to discuss more particularly in this connection some of the divergencies between scientific and technologic opinions with the view to a reconciliation. Without a certain unity and purpose little progress can be made in research work. It may be stated that the writer's recent research has been limited largely to studies of the phenomena of earth resistance. In this case, however, the anomalous behaviors encountered in studies of the phenomena of plasticity and viscosity are rather more pronounced than those which usually occur in the instances of concrete, freshly quarried rock, or green timber.

Maxwell's definitions upon the fundamental properties of matter under consideration are pertinent to the discussion; he remarks:[47]

---

[46] Prof. of Experimental Eng., Iowa State Coll., Ames, Iowa.

[47] "Theory of Heat," p. 295.

"Suppose that stresses of the same kind, but of continuously increasing magnitude, are applied to a body in succession, as long as it returns to its original form when the stress is removed, it is said to be perfectly elastic. If the form of the body is found to be permanently altered when the stress exceeds a certain value, the body is said to be soft or plastic, and the state of the body when the alteration is just going to take place is called the limit of perfect elasticity. If the stress be increased till the body breaks, or gives way altogether, the value of the stress is called the strength of the body for that kind of stress. If breaking takes place before there is any permanent alteration of form, the body is said to be brittle. If the stress when it is maintained constant causes a strain or a displacement in the body which increases continually with the time, the substance is said to be viscous. When this continuous alteration of form is only produced by stresses exceeding a certain value, the substance is called a solid, however soft it may be. When the very smallest stress if continued long enough will cause a constantly increasing change of form the body must be regarded as a viscous fluid, however hard it may be. Thus, a tallow candle is much softer than a stick of sealing wax: but if the candle and the stick of sealing wax are laid between two horizontal supports, the sealing wax will in a few weeks of summer bend with its own weight while the candle remains straight. The candle is therefore a soft solid, and the sealing wax is a viscous fluid."

From the critical viewpoint, it is natural to ask: "Is concrete a viscous fluid or a plastic solid?" According to the author's intimation concrete has no sensible elastic limit except it be that at the origin of the stress-strain curve, these curves being usually drawn continuously concave toward the axis, which is commonly taken horizontal. If concrete is a plastic solid as implied in the author's title, then according to Maxwell's definition, as mentioned, it should have a sensible elastic limit differing from that at the origin, which, however, the standard treatises do not show. This fact is tacitly admitted by the author in his citation relating to Bach's researches.

The writer has found in his own researches that if the test load is applied slowly enough on cylinders of earth taken from their natural beds, the stress-strain curves from the data are similar to those of concrete and cast iron in that they are continuously curved toward the strain axis. When the stress-strain readings, however, are taken quickly, but without appreciable impact, say, in the space of 15 to 30 sec., there is a sensible elastic limit, with the stress-strain curves ensuing in a manner quite similar to those found for steel columns.[48] On the other hand, in the case of steel columns, when considerable refinement is used in making the strain measurements, there is not, strictly speaking, a perfectly constant modulus of elasticity, nor a true elastic limit for steel. The existence of these functions is more or less a relative matter, and is conditioned to a large extent by the time-rate of application of the load. One of the great educational advantages in observing earths, apart from the immediate technologic needs, is that, being a "visco-elastic" material, the behavior of earth shows in a magnified way what may be expected to occur in other apparent matters so that an investigator can often anticipate phenomena and properly interpret them dynamically. A perfectly elastic matter, so-called, gives a specific and limited rather than a general behavior exhibiting all the properties of matter. A good illustration of that is afforded in the instance of heterotropic crystals compared with isotropic or amorphous matter, such as many soils.

---

[48] See *Technologic Paper No. 101,* U. S. Bureau of Standards, for curves.

Researches by Dr. Abram F. Joffé, of Leningrad,[49] a collaborator of Röntgen, have a direct bearing upon most of the points under discussion. His researches upon atomic and crystal structure substantiate the opinions which have long been held as to the approximate nature of Hooke's law of proportionality of stress to strain. He sustains the judgment of George Green,[50] with Cauchy, one of the recognized founders of the theory of elasticity, and of Boussinesq, who conceived the importance of developing stress functions in series as considered later. Boussinesq[51] developed many functions in series. He regarded ordinary matter as an intermediate type between the so-called perfect fluid and perfect solid. He so arranged his functions[52] that this intermediate matter would merge with the perfectly continuous matter of the theory of elasticity as an extreme case. Dr. Joffé gives many interesting statements bearing upon Hooke's law and the so-called elastic after-effect. In one instance he was in some doubt as to whether the particular type of crystal could be considered as a solid. His remarks on the relation of the elastic after-effect to hysteresis should be considered in connection with the author's interpretation of the phenomenon.

Many of Dr. Joffé's observed phenomena studied by means of the X-ray spectrometer and electrical processes may be observed upon the macrostructures as well. It is known from the strain measurements conducted at the Washington Monument by the late Dr. Becker and Dr. Van Orstrand[53] that steel is in a continual state of flux, the observations being conditioned by the intensity of loading, its rates of application, and the degrees of magnifications of the different strains. It is also a fact of experience that both green and seasoned timber flows continually an infinitesimal amount with the time. On account of this it is necessary for the U. S. Forest Service to adopt certain rates of deformation in order that test data on timber may be properly interpreted and compensated. It is shown by Withey and Aston[54] that a maximum strength of 100% on long leaf yellow pine timber, under momentary or short-time loading, will be reduced to only about five-eighths of the value with a continuous application of the loading for a period of 700 hours.

The writer has observed in his own researches that earths cut from the natural beds flow continually in almost imperceptible, but easily measurable, amounts. This flow continues for many days without reaching statical stability when the viscous sub-matrix or "binder" ceases to function from a complete "grain gearing" under the applied load. When there is complete particle gearing, twelve particles are in contact around a particle nucleus, as in the case of annealed glass, and in conformity with the structures of granular matter discussed by Dean C. S. Slichter[55] and the late Professor Osborne Reynolds[56] for the different orders of matter considered in their respective fields of investigation. In the case of experimental piles sunk into the earth there is a tendency for the pile to resume the unstrained position after the load is removed if time is given. It is also well known that when proof loads are applied to steel and cast-iron members well within the elastic limit the stress-strain curves tend to straighten conformably to Joffé's observations and presumably owing to the fact that the crystals of the metal tend to approach the configuration of maximum density already cited.

---

[49] "The Physics of Crystals," 1928, pp. 23, 33, and 44.

[50] "The Mathematical Theory of Elasticity," A. E. H. Love, Fourth Edition, p. 11.

[51] "Essai Theorique sur L'Equilibre des Massifs Pulvérulents," par M. J. Boussinesq, Bruxelles, 1876, pp. 6 *et seq.*

[52] *Loc. cit.*, Chapters I, II, and III.

[53] Unpublished Memoir, kindly loaned the writer by the Acting Director of the U. S. Geological Survey.

[54] Johnson's "Materials of Construction," Fifth Edition, p. 207.

If matter flows very slowly, but still perceptibly, like a glacier, with the phenomenon of flow occurring from the beginning of the time of stressing, it would seem appropriate, in the writer's opinion, that the behavior should be called viscous instead of a plastic phenomenon in accordance with the scientific nomenclature. It is questionable whether this flow should be regarded as hysteresis according to the author's point of view, which is the lag experienced in the case of cyclic stress and similar phenomena in other fields of science. The supposition of a truly reversible process in any of Nature's laws is, of course, a purely analytical fiction made for convenience mathematically. One may accept it as an induction of experience that there is always some degree of dissipation of energy. In the extreme case of light transmission, Professor Osborne Reynolds reached the conclusion that the viscous dissipation is sufficient to reduce the total wave energy to one-eighth in 56 000 000 years.[57]

The phenomenon of a flux of matter disappears entirely when there is an absence of the viscous sub-matrix in the particle interspaces which possesses the nature of a grain lubricant. This occurs in the case of dry sand and gravel under pressure partly from the particles getting on dead centers, as in the case sometimes occurring for locomotive drivers, and partly from the building up of frictions under loading. According to Boussinesq the particles gain an elasticity of bulk. The writer demonstrated this by loading tracing cloth cylinders, 4 in. in diameter by 14 in. high, filled with gravel and tested to destruction. The cylinders became as hard as flint as the grain frictions built up under loads of 800 to 900 lb., thus affording a good hypothesis as to why tool steels harden when heated above the critical range of temperature and are quenched in oil or water. The hoop tension supplied by the outer shell of contracting material causes a gain in bulk elasticity from the up-build of pressure and inter-crystal friction.[58]

Inasmuch as the author's criticisms of the theory of elasticity evidently relate to the theory as used by technologists, and there is often a wide discrepancy here between that and the classical theory developed by scientists and engineers, the writer will discuss some of the fundamental principles of the rigorous mathematical theory to correct some of the impressions commonly held. Hooke's law, as the author has shown, should be properly interpreted. This law should be regarded a limiting aspect. It strictly applies solely to infinitesimal strains; it does not take cognizance of finite strains as sometimes used by technologists, except to a first and sometimes a very rough approximation as the author remarks in effect. George Green,[59] in formulating his dynamics of potential, conceived that the functions could be developed in power series. The idea is clearly set forth by William H. Burr, M. Am. Soc. C. E., who remarked more than forty years ago:[60]

---

[55] 19th Annual Rept., U. S. Geological Survey, Pt. II, p. 306 *et seq.*

[56] Collected Works, Vol. III, p. 1 *et seq.*

[57] *Loc. cit.*, p. 238, Article 258.

[58] Johnson's "Materials of Construction," Fifth Edition, p. 634.

> "It is a matter of experience that strains always vary continuously and in the same direction with the corresponding stresses. Consequently, the stresses are continuously increasing functions of the strain, and any stress may be represented by a series composed of the ascending powers (commencing with the first) of the strains multiplied by proper coefficients. When, as is usually the case, the displacements are very small, the terms of the series whose indices are greater than unity are exceeding small compared with the first term, whose index is unity. These terms may consequently be omitted without essentially changing the value of the expression. Hence, follows what is ordinarily termed * * * Hooke's Law."

Since there is but one fundamental method of analytical continuation known to mathematicians, the various methods used being equivalent to Taylor's, the French mathematician, Boussinesq, proceeded along strictly logical analytical lines in his development of functions by Maclaurin's well-known formula, this being a special case of Taylor's formula, in handling the finite displacements of granular materials. A good illustration of such a development is illustrated in Love's development of strains by Taylor's theorem of continuation. For large strains he obtains expressions of the types:[61]

$$(\delta_{xx}) = \frac{\partial \mu}{\partial x} + \frac{1}{2}\left[\left(\frac{\partial \mu}{\partial x}\right)^2 + \left(\frac{\partial v}{\partial x}\right)^2 + \left(\frac{\partial w}{\partial x}\right)^2\right]$$

$$(g_{yz}) = \frac{\partial w}{\partial y} + \frac{\partial v}{\partial z} + \frac{\partial \mu}{\partial y}\frac{\partial \mu}{\partial z} + \frac{\partial v}{\partial y}\frac{\partial v}{\partial z} + \frac{\partial w}{\partial y}\frac{\partial w}{\partial z}$$

for finite normal and shearing stresses. When the strains are ultimately small in accordance with Professor Burr's description, the higher order derivatives vanish, as he states, leaving the ordinary expressions used by engineers and elasticians, or,

$$\delta_{xx} = \frac{\partial \mu}{\partial x}$$

$$g_{yz} = \frac{\partial w}{\partial y} + \frac{\partial v}{\partial z}$$

Since the author raises the question of the propriety of the usage of initial, tangent, and secant moduli, the writer also calls attention to the fact that Professor Burr gives the correct analytical expression of a modulus as the derivative of stress with respect to strain. This may be constant for a plastic material below the elastic limit; it is a variable for viscous materials generally and for plastic materials above the elastic limit. From the critical standpoint the modulus must also be defined with respect to the state of matter existing. For example, the late Professor William Kent when discussing thermo-dynamic relations in his well-known "Mechanical Engineers' Handbook", treats the subject of moduli at constant pressures and at constant volumes; but since a scientist must deal with a number of variants in defining the states of matter in conformity with such well-known laws as Willard Gibbs' phase relation, van der Waal's, Berthelot's, Keyes', and the hundred others proposed,[62] it is correct to speak of moduli at constant temperatures, at constant density, or in terms of chemical composition, three states usually being sufficient. In the theory of earth resistance the pressure and density are the important variants. In the correct usages of the theory of elasticity as distinguished from the many nascent soil theories being proposed it is necessary to use a number of variants; pressure, volume, temperature, density, and chemical variants arise as instanced in the case of simultaneous water, ice, and steam phases. Kelvin and Kirchhoff were particularly concerned with isothermal and adiabatic straining in defining the existence of a potential function.[63]

---

[59] "The Mathematical Theory of Elasticity," A. E. H. Love, Fourth Edition, pp. 11–12.

[60] "The Elasticity and Resistance of Materials," Second Edition, p. 2.

[61] *Loc. cit.*, p. 60.

The author's exponential functions for stress-strain relations and time-yields are of the nature of pioneer work; such relations have the support of C. Bach and other German authorities. G. S. Williams, M. Am. Soc. C. E., and A. N. Talbot, Past-President, Am. Soc. C. E., have made considerable use of expressions of this kind notwithstanding their empirical forms. There is a deviation here to be noticed between the author's types of empirical equations and those of the rigorous theory of elasticity. The rigorous expressions are the well-known equations of motion (or rest in particular) of the theory of elasticity.[64] They are analytically equivalent to Newton's second law, force = mass × acceleration, resolved along the particular co-ordinate axes chosen. The labor involved whether by Mr. Straub's analyses or those of the theory of elasticity is relatively enormous. The plan used extensively by the writer in laboratory measurements has been to make direct correlations of the experimental data with those of the theory of elasticity on the various types of structures studied either by the approximate or rigorous methods. One way of proceeding is to measure the functions and reduce them to empirical laws for provisional use, following Newton's plan of "no hypotheses". By the ordinary process of curve fitting, using the theory of elasticity as the limiting case, the true relations are discovered. This is similar to the curve-fitting in the theory of columns, in which Euler's theory and the short column formula are limiting cases to fulfill. The method is really what Poincare in his "Theory of Potential" calls a sweeping-out process ("méthode du balayage"); in other words it is one of successive approximations. The laws found, however, are laws in fact of the kind mentioned by Halbert P. Gillette, M. Am. Soc. C. E.;[65] they agree with the observations and satisfy the equation of motion. Attention should be paid to the fact that the author has developed the right-hand members of the equations of motions or rather the primitive deflections which must be differentiated twice in getting the accelerations, subject to the conditions that his functions must be continuous. When the functions are not absolutely continuous as often occurs in practice they are "smoothed-out" to make them so. As far as the experimental methods are concerned, that of G. E. Beggs, M. Am. Soc. C. E., which is suggestive of the older barrel-hoop method of Eads, leaves little to be desired. Love gives the kinds of equations needed for the arch.[66] Cylindrical co-ordinates are usually the simplest to use.

---

[62] Washburn's "Principles of Physical Chemistry," Second Edition, see Index.

[63] Perry's "Applied Mechanics," Chapter 15, pp. 366–367. In the author's case the strain is in effect applied at constant temperature, volume, and composition, so that the modulus should be defined at constant pressure and constant density.

[64] "The Mathematical Theory of Elasticity," A. E. H. Love, Fourth Edition, Chapter XXI, p. 451, also Chapter XVIII.

[65] *Engineering and Contracting*, March, 1929, p. 109, third column.

In working out correlations with dynamics according to the no-hypothesis method of Newton, it often helps to keep in mind the doctrine of self-similitude enunciated by his contemporary, Swedenborg.[67] As a simple example it may be mentioned that the single potential function, $\frac{1}{r}$ (r = distance from body to point), is fundamental in the theories of astronomy, earth resistance, elasticity, and hydrodynamics, in Osborne Reynold's theory of attraction, and in Dr. Joffé's theory of the atom in which direct correlations are carried out consonant with the writer's plan of no-hypotheses.[68]

Charles W. Comstock,[69] **M. Am. Soc. C. E.** (by letter).—In this paper the author has considered (and to some extent confused) two questions which, in general, are unrelated, namely, the stress-strain relation and the plasticity of materials. The material specifically referred to throughout the paper is concrete but, except for numerical values of certain constants, no reason appears why the arguments should not apply equally to all structural materials.

Rather more than one-half the paper deals with "the proposed theory of elasticity" and illustrative applications. Here, the author has steered wide of Scylla only to be engulfed by Charybdis. He rejects the linear stress-strain relation, but accepts without question the old Bernouilli hypothesis of conservation of plane sections, which not only has no theoretical or experimental justification, but is known to be incorrect except where there is no shear.

This important point is treated very casually. The author attempts no justification of the hypothesis, nor does he write as if he regarded it at all seriously. For example, he states:

"From Fig. 6(*b*), it may be seen that, assuming conservation of plane sections during bending,

$$\varepsilon_z = z\,\frac{d\,\phi_E}{d\,s} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)\text{''}$$

There is no other reference to it in the text. Figs. 6(*b*), 8(*a*), 16(*b*), and 17(*b*), indicate that the deformation is considered as a simple rotation of plane sections about their respective axes, the positions of which are to be determined. As to Fig. 16(*b*), the assumption is probably correct since there is no shear; as to the others, and, in general, it is not true.

---

[66] "The Mathematical Theory of Elasticity," A. E. H. Love, Fourth Edition, p. 451.

[67] Emerson's "Representative Men," Lecture III, "Swedenborg."

[68] Peirce's "Newtonian Potential Function"; Lamb's "Hydrodynamics," Third Edition, Chapter V; Max Born's "Constitution of Matter," p. 56 *et seq.*; and Joffé's "The Physics of Crystals."

[69] New York, N. Y.

Having selected this starting point the author has thenceforth followed the conventional and usual routine for the analysis of arches, merely substituting for the linear stress-strain relation the exponential relation which was first suggested by Bach as a consequence of his experiments on cast iron.

There is no occasion to comment on the algebraic transformations which lead ultimately to Equations (32). These can contain no more truth than does the assumption on which the entire analysis is founded. It would seem that the author has been misled by the increased complexity of his formulas into expectation of correspondingly greater precision. This is a not infrequent misconception.

The "proposed theory of plasticity" is a repetition of the proposed theory of elasticity with the introduction of a time factor which, however, runs unchanged through the entire analysis to re-appear in Equations (62) just as originally introduced in Equation (34).

It is not clear why the "elastic" and "plastic" deformations should have been separated. Assuming the stress-strain relation known it is of no importance whether or not the whole or any part of the strain disappears on removal of the applied forces. The stress distribution is the same in any case.

The expression, "theory of elasticity," originated at a time when somewhat limited experimental data indicated that small strains were elastic in the ordinary sense of the word, and that such strains were directly proportional to the respective stresses.

The mathematical theory of elasticity is a highly developed branch of mathematical physics, singularly free from the speculations and incongruities which permeate so much of modern physical theory. Its fundamental differential equations rest directly on the laws of statics and are no more questionable. In passing from stress to strain it is always carefully stated that linear relationship is assumed. No other assumptions are made. The theory has been extensively and intensively developed through the past hundred years and has been logically set forth in many scholarly treatises in all languages, so that there is now no reason why any interested person should be unfamiliar with its possibilities or its limitations. It is equally true whether or not there exists any real material for which the stress-strain relation is linear.

Most of the alleged defects of this theory are chargeable, not to inadequacy of the theory itself, but to its attempted application outside its ostensible field or, more often, to the injection of unjustified and unjustifiable hypotheses.

If one has to deal with a material for which the stress-strain relation is known to be other than linear, the difficulties are enormously increased, but no progress can be made by the assumption of stress or strain distributions which do not fulfill the fundamental conditions of the problem.

The equations applicable to a particular problem are sometimes extremely involved. Many years ago the writer convinced himself that, at least in part, this was because of the large number of variables resulting from the use of three components of each magnitude. In view of the directional character of both stress and strain it seemed that quaternion analysis should be an improvement over the usual method of Cartesian co-ordinates. This the writer attempted with some degree of success. The results were not all that were hoped for, but they were encouraging, and it is still believed that Sir William Rowan Hamilton's brilliant conception will afford solutions which have baffled the more common mathematical methods.

It is not clear why the author should have selected 1.3 as his value of m. Mr. Straub states: "Data obtained from typical experiments by Dr. E. Probst have been used by the writer as a basis for analysis in the basic equation * * *." He then gives equations in which m for compression is 1.105, and for tension, 1.03; yet in all his numerical illustrations he uses $m = 1.3$.

Likewise with respect to p; from the experiments of Mr. Earl B. Smith he finds $p = 1.73$, while from the observations of Freyssinet he concludes, after making an assumption as to shrinkage during setting, that $p = 1.25$. However, he uses 1.3 for numerical illustrations.

As regards q, he finds from the work of Messrs. Fuller and More the value, 0.42, from Smith's experiments, 0.18, and from Freyssinet's observations, after a questionable allowance for shrinkage, 0.40.

Unless the author has a mass of experimental data not cited in his paper, his choice of numerical values would appear to rest on a rather flimsy foundation. If he has such data, full presentation of them, together with the reasoning process by which he determines the most probable values of the constants for use in his formulas, would certainly be of quite as much interest as the formulas themselves.

The form of the author's Equation (34) is open to question. In accordance with it, deformation under any stress whatever must increase progressively for all time, and should end in failure of the structure. It is doubtful whether this conclusion will find general acceptance.

Even if it is true that strain is a function of both stress and time, it is not clear on what ground the author justifies the assumption that it may be expressed as the product of two functions, one of stress only and the other of time alone.

For concrete, of which the properties certainly change with time, at least during its early history, a more probable assumption would be Equation (33), in which, m should be a function of the time. The form of the function, m, should be such that it would become practically constant after a finite lapse of time.

It is doubtful if there exist to-day sufficient data of adequate precision to justify an attempt to formulate the stress-strain-time relation for concrete.

The conclusions drawn by the author are based on his formulas. These, in turn, rest on the false hypothesis of conservation of plane sections and on insufficiently established stress-strain-time relations. The conclusions may be true, but they are not proved.

A. A. Eremin,[70] Assoc. M. Am. Soc. C. E. (by letter).—This paper, by giving consideration to the natural behavior of concrete, is a step forward in the design of concrete arches. The subject is well treated and the development of the formulas is clear and can be easily followed. There are, however, a few phases that could be discussed and possibly extended.

---

[70] Asst. Designing Engr., Bridge Dept., State Highway Comm., Sacramento, Calif.

It has been known for a long time that strain is not proportional to stress in such materials as concrete, stone, and cast iron; or, in other words, they do not follow Hooke's law. In 1896, C. Bach[71] published a paper on the variable modulus of elasticity for concrete at different stresses. Many attempts have been made since that time to develop the formulas for structural members with variable moduli of elasticity. Even for a beam with simple loading, the time required for the calculation of the stresses by means of these formulas, was prohibitive. Bach[72] proved that, in the formulas containing the variable moduli of elasticity, the position of the neutral axis varies with the stresses and also with the change in the external concentrated loads and bending moments.

Max Ritter[73] analyzed an arch by the ordinary elastic method with the constant modulus of elasticity and also by the method with the variable modulus of elasticity, using the exponential expression for strain similar to that used by the author. In conclusion, he stated that for the arch in which no tension occurs there is no practical difference in the stress obtained whether a constant or a variable modulus of elasticity was used. This supports the author's conclusion that "for smaller structures the usual procedure for stress analysis is probably satisfactory". It is interesting to note that Ritter in his study of arch stresses limited his conclusion to the arches in which no tension occurs because, like Mr. Straub, in the formulas for variable modulus of elasticity, he assumed that the exponential expression for strain of concrete in tension is the same as in compression. This assumption greatly simplifies the calculations although, at the same time, the formulas developed cannot be used for pure bending stresses without appreciable error.

The actual stress diagram in the plain concrete in pure bending will be shown by the curved line, *AB*, Fig. 31.[74] When a constant modulus of elasticity is used the stress-strain line will be the straight line, *A'B'*, and the neutral axis will pass through the center of gravity of the section (Fig. 31).

The stress-strain diagram for concrete varies with the grade of cement, aggregate, water ratio of mix, the method of pouring of green concrete, and the age of the concrete. Various curves for the stress-strain diagram due to these factors will be similar, but will show a marked variation in tension and compression, Line *AB* (Fig. 31).

Some of the European specifications adopted the straight-line variation of the moduli of elasticity of concrete, as shown by the lines, *A''O* and *OB''* (Fig. 31). This is somewhat nearer the actual stress-strain diagram and the constant ratio of the moduli of elasticity in compression and tension, simplifying the formulas for stresses. In the general study of elasticity and plastic flow of concrete under pure bending stresses for which numerical values are not essential, the assumption could be made that the modulus of elasticity of concrete in tension varies at the same rate as in compression, since this permits considerable reduction in the formulas.

---

[71] "Hütte," des Ingenieurs Taschenbuch, 1, 1920, p. 500.

[72] "Elasticität und Festigkeit," 1911, p. 236.

[73] *Schweizerische Bauzeitung*, January, 1907, p. 25.

[74] "Strength of Materials," by Prof. S. P. Timoshenko, Russian Edition, 1916, p. 234.

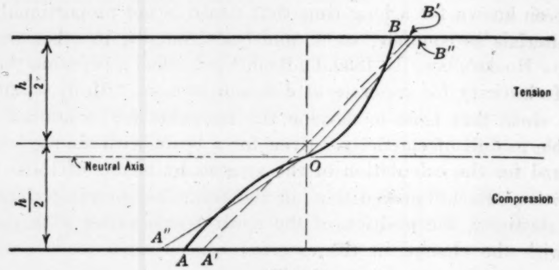


FIG. 31.

In the author's "First Conclusion" he states that:

> "It is incorrect to assume that in general plastic flow in statically indeterminate concrete structures causes a redistribution of bending moments tending to relieve the overstressed parts of the structures."

The overstressed part of a statically indeterminate structure is distinguished by excessive deformation which will affect other parts of the structure similar to some external disturbances which were discussed by the author in his "Second Conclusion".

Take, for instance, a beam with one end fixed and the other end simply supported, as shown by the author in Fig. 21. The overstressed part in the middle of the span will produce excessive deflection of the beam, and, at the same time, the reversed bending moment at the fixed end will be increased. Due to plastic flow, a redistribution of the stresses will occur, similar to that caused by the settlement of a support.

Kern analyzed a concrete arch with fixed ends and found that the resultant stresses due to dead load, rib-shortening, and temperature changes, computed by his method[75] with variable moduli of elasticity, are smaller than those computed by the ordinary method with constant moduli of elasticity. He thus proved the author's "Conclusion" that temperature "stresses computed by the ordinary elastic theory are * * * on the side of safety."

In conclusion, it may be stated that Mr. Straub has made a valuable contribution to engineering literature, and that further theoretical and experimental study of elasticity and plastic flow of concrete arches is highly desirable.

DR. ING. J. MELAN[76] (by letter).—In applying the elastic theory based on Hooke's law to the design of concrete structures, it is recognized that only approximate values of deformations and stresses are obtained. The assumed proportionality between deformation, or shortening, and stress actually does not exist in materials used for masonry structures: First, because the ratio of stress to strain is not a straight-line relation; that is, the modulus of elasticity is not constant for all stress intervals; and, second, because permanent plastic deformations occur which are dependent on the age of the concrete and the period of loading. The inaccuracy of computation is less, however, the longer the concrete is allowed to set before it is loaded.

---

[75] *Beton u. Eisen*, 1930, p. 29.

[76] Tech. Univ. of Prag, Prag, Czecho-Slovakia.

It is not easy to set up mathematical laws accounting for the stress-deformation relations of concrete. Hence, the treatment given this subject by Mr. Straub who proposes such relations, is of particular interest.

As a basis for elastic deformation, he uses the Bach-Schüle law, $\epsilon = k\ \sigma^m$, and for plastic deformation he develops another equation of the exponential type as follows, $\rho = K\ \sigma^p\ t^q$, in which, the deformation is a function of the stress, σ, and the time, t. According to available data for time-yield experiments, although these are few, this assumption seems to define the law of plastic deformation for concrete quite well. The coefficient and exponents are dependent on the composition of the concrete and its age.

The theory derived for fixed-ended continuous arches based on the foregoing equations is mathematically sound, assuming that plane sections remain plane and if the arch undergoes only compressive stresses, and no tensile stresses or only small ones, as a result of the loading and deformation. With these stipulations, the conclusions drawn by the author are also correct. However, the conclusion that the early removal of centering from a reinforced concrete arch is advantageous to the structure, cannot be accepted without reservation. It has not been demonstrated in practice, and conflicts with the rules of construction maintained to the present time, deviation from which may prove to be a dangerous venture.

In any case, Mr. Straub's treatment provides a valuable and meritorious supplement to, and a more thorough analysis of, the theory of concrete arches.

FREDRIK VOGT,[77] ASSOC. M. AM. SOC. C. E., and HERBERT J. GILKEY,[78] M. AM. SOC. C. E. (by letter).—The author concludes his paper with the words:

"Much more experimental work, especially of a quantitative nature, is desirable and necessary in order to establish definitely the laws of plastic flow for concrete of various mixes and ages."

This is doubtless true, and for that reason the development of an imposing mathematical analysis upon assumptions of questionable validity appears to be premature at this time. Important problems regarding the qualitative as well as the quantitative nature of flow are still unsolved.

To what extent, for example, is flow plastic and to what extent is it elastic? Most structural materials have a distinct elastic recovery when a stress of any intensity is released. Professor F. B. Seely[79] alludes to the energy of this elastic recoverance as hyper-elastic resilience, as distinguished from elastic resilience (to which the term, resilience, is generally applied). Concrete has this same power of partial recovery within all ranges of stress. That it exists to an appreciable extent, there is no question.

For dead-load stresses the elastic recovery is of no importance, since there will be no release of load; but it is important in connection with stresses due to temperature changes in statically indeterminate structures since these changes have an annual or a seasonal period.

---

[77] Docent, Norges Tekniske Höiskole, Trondhjem, Norway.

[78] Prof., Civ. Eng., Univ. of Colorado, Boulder, Colo.

[79] "Resistance of Materials," pp. 15 and 361.

From observations too limited to generalize other than to state that appreciable recovery is a reality, the writers have found that concrete, after unloading, will immediately recover an amount about equal to the initial elastic deformation. In addition to this first recovery there seems to be a slow additional recovery of part of the flow.

Another unsettled problem is the connection between volumetric changes (shrinkage and swelling) due to moisture changes and the flow due to load. The author states that:

> "Shrinkage is distinguished from plastic flow in that it is a change in dimensions independent of external forces acting on the specimen."

Let the change in length of an unloaded specimen at constant temperature be termed shrinkage (or swelling). Then, the difference between this shrinkage and the change in length of a specimen exposed to a continuous load is termed flow. The writers know of no evidence that the flow determined in this way is independent of the shrinkage, or, to go a step further, independent of moisture changes and changes in the fine structure of the concrete, such as re-crystallization, etc. On the contrary, there is some reason for suspecting that there may be a connection between the two phenomena. It is a striking fact that concrete specimens under continuous compressive loading, stored in air, flow several times as much as if stored under water.[80] The writers have described, elsewhere,[81] the result of their own experimental work on the subject. In air, the shrinkage and the compression due to the load co-operate in reducing the length, while in water the swelling under water counteracts the compression due to load. The influence of the load taken separately, that is, the flow, is several times as great in the case of co-operation with shrinkage as it is when counteracted by swelling, even after the direct shrinkage and swelling (as determined from unloaded specimens) have been deducted.

While the difference is due in part no doubt to the difference in physical condition, it seems possible and probable that the loaded or unloaded state in combination with the dry or wet condition may have a distinctive influence upon such internal phenomena as re-crystallization. Attempts have been made to supplement the available information by duplicate tests under continuous tensile loading in both air and water. The tensile flow tests have not been satisfactory on account of the low range of strain that is available in tension. It is to be noted that, with moduli of elasticity nearly equal in tension and compression, and the ultimate tensile strength about one-tenth the ultimate compressive strength, the tensile deformation available is only about one-tenth of that for compression.

Bending of beams under continuous loading was easier to handle and gave results that were in all respects similar to those obtained in compression. The beams represent a mixed condition with both tension and compression present, and the results are, therefore, more difficult to interpret.

---

[80] See the experiments made by R. E. Davis, M. Am. Soc. C. E., Report on Arch Dam Investigation, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 211.

[81] *Proceedings*, Am. Soc. for Testing Materials, Vol. 29, Pt. II, 1929, p. 706.

If the flow and shrinkage or swelling are interrelated and interdependent, the fact will have an important bearing upon the stresses due to flow. This is particularly true in bending and where temperature alternates the stresses. It also has a bearing on the question of recovery.

The author has based his deductions on a formula for the total deformation,

$$\varepsilon_{\text{total}} = \varepsilon + \rho = k\,\sigma^m + K\,\sigma^p\,t^q \ldots\ldots\ldots\ldots\ldots(116)$$

as an extension of the Bach-Schüle exponential formula for the initial deformation given by the first term.

The Bach-Schüle formula gives a fair approximation to the stress-strain diagram for high stresses, but it is well known that it is not valid for low stresses. For example, with σ approximating zero, the exponential formula gives moduli of elasticity, E, approaching infinitely large values if m is greater than unity, as it is for concrete.[82] This result is not in agreement with experience and greatly limits the formula in its application. Similar limitations are probably applicable to the second term. It is more important that the exponential term, t^q, with q positive, shall give an ever-increasing flow which approaches no limit or stopping point. The factor, t^q, reaches infinitely high values with increasing age. As a result, an arch bridge, according to the formula, would deflect until it was flattened to the point of failure. Obviously this does not happen, and the law must be different even if the formula gives a fair approximation during the first few weeks or months. In Fig. 14 the check with some experimental results is shown for a period of less than five months, but this does not justify the use of the formula for a period of more than nine years, as was done in Fig. 24. Nor has the author limited his analysis to nine years; he has extended it until the time interval becomes infinite as regards the point of contraflexure for the beam with which he deals. In other words, he has extended it until the deflection, according to his formula, also becomes infinite.

The author's "Second Conclusion" states:

> "If there is an external disturbance * * *, such as the settlement of a support, the plastic flow causes a gradual redistribution of stresses such that the structure approaches (but never reaches) its original state of equilibrium (no settlement of support)".

This conclusion results from assuming a flow which will cause the modulus of elasticity to approach zero as time goes on. With a secant modulus,

$$E = \frac{\sigma}{\varepsilon} = \frac{\sigma}{k\,\sigma^m + K\,\sigma^p\,t^q} = \frac{1}{k\,\sigma^{m-1} + K\,\sigma^{p-1}\,t^q} \ldots\ldots\ldots(117)$$

and, with q positive, E approaches zero as t becomes infinitely great. Thus, the deformation due to a given load increases indefinitely with time, and the effect of initial external disturbances becomes smaller and smaller in comparison with the increasing strains from the regular or primary loading. Finally, they become relatively so small that their influence may be considered to be negligible.

---

[82] Compare, for instance, the German Handbook, "Hütte," Vol. I, Twenty-fifth Edition, p. 541.

Conclusion 2 can only be reached on the basis of the assumed formula for flow. If the flow does not actually increase indefinitely, but if instead it approaches a limiting value, E will remain a finite value, and the result will be entirely different. If, for example, the flow increases the deformation to three times the initial value, then the influence of the settlement of a support on the stresses will be reduced to one-third of what it was at first, but not to zero since the deformation due to the settlement can be eliminated by one-third of the initial force. Compared with this major result the influence of small deviations from the straight stress-strain diagram becomes unimportant.

As stated by the author, the temperature changes and shrinkage are among those external disturbances the influence of which is reduced by the flow. However, this is only true for slow temperature changes and not for rapid or moderately rapid changes. Of course, annual fluctuations cannot receive the benefit of the flow during more than a few months since their action is reversed.

That the flow does decrease with time is shown quite conclusively by the Davis and Troxell experiments.[83] These tests show that the flow is asymptotic and that it follows the exponential formula expressed by the author for only a short time. This result is also supported by the experiments of the writers, of which previous mention was made.

The experiments by Professor Davis, already mentioned, indicate that the tendency of flow is to straighten out or flatten the stress-strain diagram where the initial deformation shows a modulus of elasticity, E, decreasing with increasing stress, or where the exponent, m, is greater than unity in the Bach-Schüle formula. In Table 28[84] of the report of the Committee of Engineering Foundation on Arch Dam Investigation, Column (10) gives the ratio of flow to initial elastic deformation. Within each group the ratio is, generally speaking, lower for high stresses than for low ones and the original curving of the stress-strain diagram (decreasing E with increasing stress) is counteracted to some extent by the flow. The effect of flow is to reduce to a smaller, but more constant, value of E. Experiments made by the writers support these findings. If they are correct, the conception of stresses and changes of stresses due to flow and the effect of flow on the stresses can be stated rather simply: If the flow gives a total deformation approximately proportional to the stress and greater than the initial deformation (that is, if E is lowered by the flow and the stress-strain diagram is straightened out), then:

(*a*) The errors in the usual computations caused by the fact that the stress-strain diagram is not straight, as usually assumed, are reduced by the flow. Particularly in compression, such as in arched bridges, the original error is comparatively small since the stress-strain diagram is fairly straight for compressive working stresses, and the error is on the side of safety.

(*b*) Stresses due to slow temperature changes and shrinkage are reduced due to the flow. They can be computed approximately by assuming a constant value of the modulus of elasticity, but this modulus must include the initial deformation and the flow due to temperature forces; that is, it is lower than the modulus for short-time tests. The same applies to stresses due to slow settlement of supports. The stresses due to load only are not dependent upon the modulus of elasticity if the stress-strain diagram is straight.

---

[83] *Proceedings*, Am. Soc. for Testing Materials, Vol. 29, Pt. II, 1929, pp. 695–699.

[84] *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 213.

With respect to Conclusion 4 it should be mentioned that the straightening of the stress-strain diagram gives the same action as when $m > p$ in the author's notation and this will produce, as he has stated, an effect just opposite to that which he has found. This will also make the author's Conclusion 3 rather doubtful.

As the discussion must imply, the writers consider the paper to be misleading in its present form. Nevertheless, they do feel that the concept is excellent. If, as stress and flow phenomena relating to concrete become better understood, a similar analysis based upon a sounder premise could be made, there is no question but that it would be a contribution to the subject. Already, much additional information has been contributed[85] toward the fuller and better understanding that should be the basis for such an undertaking.

Dr. Ing. E. Probst[86] (by letter).—The very interesting theoretical investigations by Mr. Straub have as their purpose the development of a method of computing stresses in concrete arches which conform to the actual conditions more closely than those assumed in applying the usual elastic theory based on Hooke's law.

It has long been recognized that concrete does not possess a linear relation between longitudinal deformation and stress. Furthermore, it has been known that, for concrete, plastic deformation occurs which is a function of both stress and time. Mr. Straub evaluates these relations from experimental data compiled by various experimenters and expresses the elastic deformation by the exponential formula, Equation (9). He expresses the plastic deformation as a function of two independent variables, the stress and the time, in the form,

$$\rho = K \sigma^p t^q \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (118)$$

The exponents, m, p, and q, in these equations, are dependent upon the age as well as upon other properties of the concrete. Mr. Straub recognizes that, in assuming these values, considerable uncertainty exists, but believes that as a result of systematic investigations it would be possible to pre-determine these parameters with considerable certainty.

Particular care must be exercised in evaluating experimental results because frequently the investigations do not account for all influencing factors satisfactorily, and subsequent computed elimination of such factors as change in length due to shrinkage or temperature variation may lead to deceptive conclusions regarding the magnitude of longitudinal deformation.

Experience has proved that it is practically impossible to fabricate in the field concrete which has uniform properties. Nevertheless, the static computation for the structure must be completed before erection begins because the dimensions of the structural members are dependent on such computations. Thus, properties of the fabricated concrete must be assumed, which, in the most favorable circumstances, are only mean values. Computed stresses, therefore, are not common with those actually occurring in the completed structures even though careful application is made of the laws which relate, respectively, deformation to stress and stress to time. In addition, there is frequently a distortion of the stress condition as a result of cracks in the concrete which are often unavoidable.

---

[85] See, for example, *Proceedings*, Am. Soc. for Testing Materials, Vol. 29, Pt. II, 1929, pp. 695–699.

[86] Prof., Technische Hochschule, Karlsruhe, Germany.

Finally, it is assumed that the stress-strain relations are the same for concrete in tension as in compression, although it is recognized that this is not true in fact. The actual relation between the external and internal forces of an arch are set up by introducing Equations (9) and (118). It is thereby possible to compare the deviation of computed stresses by the proposed method with those by the customary method of computation.

The application of the elastic and plastic theory proposed by Mr. Straub is to be recommended in the design of structures of unusual dimensions and which justify a more complicated and time-consuming method of computation in order to obtain an actual conception of the stresses.

Under normal circumstances the method of computation based upon the usual theory of elasticity will suffice, although it will give only approximate values because of the approximate relations assumed, such as Hooke's law, the law of superimposition, Maxwell's reciprocal theorem, and others.

In his "Conclusion", the author correctly points out that the frequent assumption made with regard to statically indeterminate structures, namely, that as a result of "plastic flow" a displacement of moments takes place tending to relieve overstressed regions, is incorrect. Equalization of the stressed condition in a manner resulting in relief of the overstressed parts of the structure, so that the resistance of the material in the parts originally less stressed carries a larger load, can occur only with a structural material having properties such as steel.

In the writer's opinion these very valuable scientific investigations by Mr. Straub serve to provide a more thorough conception of the stress conditions of concrete structures. They supply a valuable supplement and amplification to the theory of arches.

PAUL BAUMAN,[87] M. AM. SOC. C. E. (by letter).—The introduction of Bach's exponential formula in the fundamental equations of internal equilibrium of a concrete structural member leads to the results shown in Figs. 9 to 13, inclusive. In assuming the exponent, m, to be 1.30, the author exaggerates the influence of the commonly committed error due to m = unity in concrete design; but his elaborate paper is undoubtedly of greater interest from a theoretical than a practical standpoint, thereby making any limit between minus and plus infinity permissible. In this connection it may be noted, however, that the exponential formula, Equation (9), is meant to be an interpretation of experimental results rather than a physical law, as the answer that k becomes infinitely great for an infinitely small stress, for example, is doubtlessly wrong. Furthermore, it is evident that the elastic behavior of concrete is not fully described by the exponential formula, as Poisson's ratio is also variable, thereby requiring supplementary information.

---

[87] Chf. Designer, Quinton, Code & Hill—Leeds & Barnard, Engrs. Consolidated, Los Angeles, Calif.

Both the linear and the exponential relations between stress and strain are approximations, the latter merely being a closer one, provided the exponent, m, is correctly chosen or experimentally determined. The merit of the first one is that it errs on the side of safety. As long as the physical process of deformation due to stress is as little explored as it is to-day, particularly with a material as erratic as concrete, it seems advisable to keep on erring by the application of Hooke's law and the law of superposition in structural design.

The tendency of self-preservation in concrete if subjected to a bending test is evident as well as in any other crystalline conglomerate. Immediately before failure occurs lasting deformations take place, starting at the extreme fibers and gradually moving toward the neutral axis. The stress distribution then corresponds to a relatively great exponent, m, resulting in an increase in stress areas as compared to the triangular distribution, thereby permitting the section, relatively speaking, to resist a greater moment under smaller maximum stresses. Simultaneously, the neutral axis, due to a difference in strain between tension and compression, which becomes more pronounced near the point of exhaustion of tensile strength, will shift from the axis of gravity so as to disengage the critical fibers somewhat on the tension side.

Recent research on crystals at the Institute of Technology at Pasadena, Calif., has probably shed more light on what is going on when deformation takes place than any similar work heretofore. According to these findings crystals are built up of small elements or blocks, each consisting of several molecules which undergo translatory and rotary motions when deformation takes place, thereby governing the behavior of the crystal. These elements are either plastic or brittle and form the basis of classification of crystals into: (1) Macroscopic plastic and microscopic plastic; (2) macroscopic plastic and microscopic brittle; (3) macroscopic brittle and microscopic plastic; and (4) macroscopic brittle and microscopic brittle.

Needless to say these findings could not be directly applied to such complicated crystalline conglomerates as concrete, but they are indicative nevertheless of what is going on when deformation takes place. In all probability concrete would be classified as macroscopic plastic and microscopic brittle; that is, the elastic limit of elementary blocks coincides with the breaking strength.

In view of the uncertainty of the tensile strength of concrete and of the steadily growing tendency to reduce the size of structural members with a simultaneous increase in span, it will perhaps be a relatively short time only before plain concrete will be barred from use in members subject to failure due to tensile stresses originating from any cause whatsoever. One cannot set a safe limit to tension in plain concrete if there is a possibility of serious vibrations affecting it, which, unfortunately, is the case everywhere on earth.

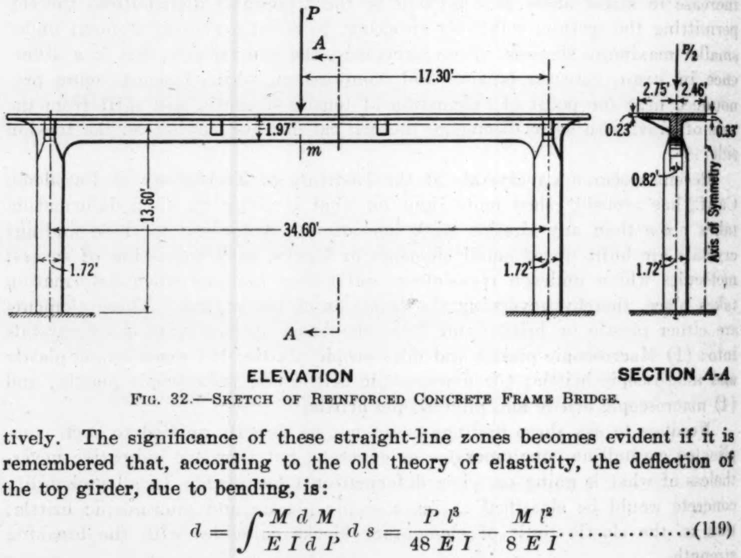
In 1903, a reinforced concrete frame bridge, as shown more or less diagrammatically in Fig. 32, was built for a National Exhibition in Dresden, Germany, and subsequently was subjected to a test[88] resulting in its failure. A concentrated load varying from zero to a little more than 77 500 lb. (Fig. 33) was applied at the center of the girder span, and records were kept of the vertical deflection, *d*, of these girders directly under the load. The strain, E_s, in the reinforcing bars directly under the load was measured and from these values the steel stresses were computed. Finally, the deformation of the posts, that is, the angular deflection at the top, horizontal deflection at center, and settling of bases, was observed.

---

[88] Dr.-Ing. W. Gehler, Betonbeiblatt No. 9, *Deutsche Bauzeitung*, 1904.

As in this discussion the change of modulus of elasticity with the change of loading is of prime interest, it will suffice to confine these studies to the top girder only.

In Fig. 33 the deflections, *d*, and the corresponding steel stresses are given for *P* varying from zero to 77 500 lb. Lines *A*, *B*, *C*, *D* and *A*, *B′*, *C′*, *D′* very closely approximate the observed deflections and steel stresses, respectively. The significance of these straight-line zones becomes evident if it is remembered that, according to the old theory of elasticity, the deflection of the top girder, due to bending, is:



ELEVATION

SECTION A-A

FIG. 32.—SKETCH OF REINFORCED CONCRETE FRAME BRIDGE.

$$d = \int \frac{M\,d\,M}{E\,I\,d\,P}\,d\,s = \frac{P\,l^3}{48\,E\,I} - \frac{M\,l^2}{8\,E\,I} \qquad \text{(119)}$$

or that it is equal to the deflection, due to $M_0 = \frac{P\,l}{4}$, of a simple beam of equal span, minus the influence of the corner moment, $-M$. Substituting $\mu\,M_0 = \frac{\mu\,P\,l}{4}$ for $-M$:

$$d = \left(\frac{2 + 3\,\mu}{96}\right)\frac{P\,l^3}{E\,I} = C\,\frac{P\,l^3}{E\,I} \qquad \text{(120)}$$

because μ is, for one and the same frame structure, a constant. This is the equation of the curve through Lines *A*, *B*, *C*, and *D* (Fig. 33) and,

$$\tan\alpha = \frac{d\,(d)}{d\,P} = C\,\frac{l^3}{E\,I} \qquad \text{(121)}$$

is its slope. Since this slope is constant between Lines *A* and *B*, *B* and *C*, and *C* and *D*, respectively, it follows that the product, *E I*, is also constant within each of the three zones. The physical meaning of this is that in Zone *A-B* the full concrete section is active on the tension as well as the compression side of the neutral axis; in Zone *B-C* the first hair-cracks occur in the concrete under tension, partly eliminating its activity; whereas, in Zone *C-D*, the limit of proportional deformation of steel is reached and failure begins. As the rigidity of the structure inside each zone remains unchanged ($\tan \alpha =$ constant), the product, *E I*, is an index of rigidity. To determine *E* of the compound material, that is, for concrete and steel or reinforced concrete, Equation (121) may be written, $E \tan \alpha = \frac{C\, l^3}{I}$.

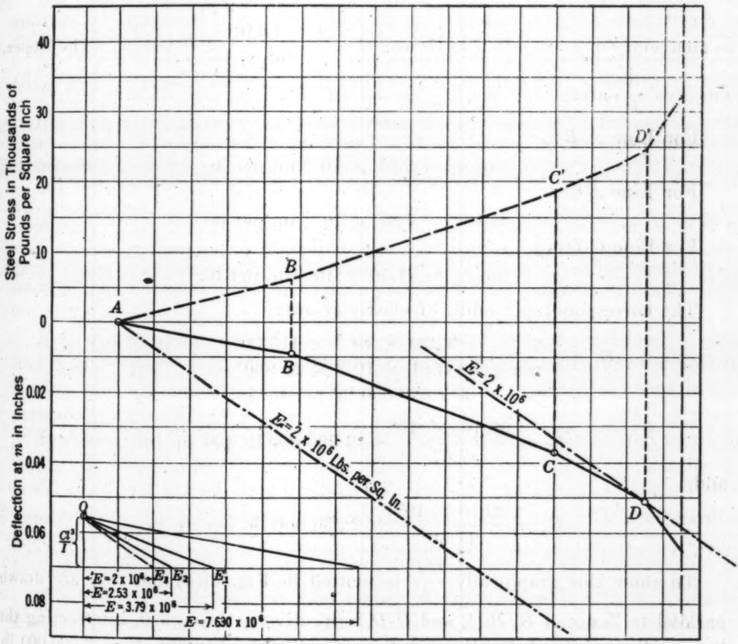


FIG. 33.—DEFLECTIONS AND STRESSES IN REINFORCED CONCRETE BRIDGE FRAME (FIG. 32), DUE TO CONCENTRATED LOADS AT THE CENTER.

In Fig. 32, $I = 22\,930$ in.^{4}; that is, the moment of inertia of the full concrete section of the top girder; $l = 34.60$ ft. $= 415.30$ in.; and $C = \frac{2 + 3\,\mu}{96}$. Using Gehler's "short-cut" for the determination of μ,

$$\mu = \frac{h\, I}{l\, I_p} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (122)$$

in which, $I_p = 15\,140$ in.[4] and $h = 13.62$ ft. $= 163.40$ in. Therefore,

$$\mu = \frac{163.40 \times 22.930}{415.30 \times 15\,140} = 0.597$$

As there is always some uncertainty as to the degree of restraint of the posts at the base, the average between fully restrained and fully hinged is taken.

For full restraint:

$$\rho_1 = 2 + \mu = 2.597 \text{ and } -\mu_1 = 0.385 = \frac{1}{\rho_1}$$

whereas, for a fully hinged base,

$$\rho_2 = 3 + 2_\mu = 4.194 \text{ and } -\mu_2 = \frac{3}{2\,\rho_2} = 0.358$$

or an average of $\mu = -0.372$. Hence, $C = \dfrac{2 - 1.116}{96} = 0.00922$. The slopes, $\tan \alpha = \dfrac{d}{P}$, are:

For Line A-B:

$$\tan \alpha_1 = 3.81 \times 10^{-6} \text{ in. per lb.}$$

For Line B-C:

$$\tan \alpha_2 = 7.67 \times 10^{-6} \text{ in. per lb.}$$

For Line C-D:

$$\tan \alpha_3 = 11.50 \times 10^{-6} \text{ in. per lb.}$$

The corresponding moduli of elasticity are:

$$E_1 = \frac{C\,l^3}{I \tan \alpha} = \frac{9\,220 \times 10^{-6} \times 415.30^3}{3.81 \times 10^{-6} \times 22\,930} = \frac{29.10 \times 10^6}{3.81}$$
$$= 7\,630\,000 \text{ lb. per sq. in.}$$

$$E_2 = \frac{29.10 \times 10^6}{7.67} = 3\,790\,000 \text{ lb. per sq. in.}$$

and,

$$E_3 = \frac{29.10 \times 10^6}{11.50} = 2\,530\,000 \text{ lb. per sq. in.}$$

To show this graphically $\dfrac{C\,l^3}{I}$ is plotted in Fig. 33(*a*) and rays are drawn parallel to Zones *A-B*, *B-C*, and *C-D*, respectively, through 0, intersecting the base at the values of E_1, E_2, and E_3, respectively. Finally, $E = 2\,000\,000$ lb. per sq. in. is plotted, and a parallel to the corresponding ray is drawn as a tangent to the Curve *A*, *B*, *C*, *D*. The point of contact approximately coincides with the end of Zone *C-D;* that is, with the beginning of failure, thereby justifying the usual assumption of $E = 2\,000\,000$ in concrete design as it approaches the lower limit. This value, however, leads to an exaggeration of deflections as indicated by a parallel line through Point *A*. At the end of the first zone, that is, at Point *B*, the tension in concrete for a load of 23 800 lb. was computed and found to be 350 lb. per sq. in., or approximately one-tenth the ultimate compressive strength of concrete. The maximum compressive stress at the center of the span, due to the ultimate load of 77 500 lb. was computed at 800 lb. per sq. in., whereas near the support it was in excess of 1 800 lb. per sq. in. Failure, however, occurred in the steel at *m* (Fig. 32).

It would be interesting, indeed, to have comparative data based on the author's new theory of elasticity.

G. C. STAEHLE,[89] ASSOC. M. AM. SOC. C. E. (by letter).—In a study of the effects of time yield or plastic flow upon concrete arches the following items are of outstanding importance:

(*a*) The modulus of elasticity of concrete, as determined by the ordinary quick loading of laboratory practice, increases progressively with age over a period of several months or more.

(*b*) The modulus decreases with age under sustained loading,[90] the amount and rate of decrease being functions of the ratio of applied unit stress to ultimate unit strength.

(*c*) The rate of advancing deformation with time for concrete under sustained loading is also a function of the ratio of applied unit stress to ultimate unit strength. (Experiments[91] by G. A. Maney, M. Am. Soc. C. E., indicate that, within the limits of working stress, the increase of deformation with time, commonly attributed to sustained load, is, in reality, due principally to shrinkage.)

(*d*) Repeated applications of stress, within the ultimate strength, each unloading being followed by a period of rest—even for a very short period—raise the proportional limit of concrete in much the same manner as that of steel and other metals. A plausible and very interesting explanation of this phenomenon has been previously given by Mr. Turner. The stress-strain relation thus becomes a linear one up to a relatively high percentage of the ultimate strength for this condition.

Bearing these four major points in mind, the ratios of dead load to total load, and of dead load and total load to ultimate strength, have a definite bearing on the amount of plastic flow which may be expected to occur in any particular structure.

It should also be borne in mind that plastic flow is accentuated by rapid drying and retarded by moist curing. This effect is influenced, in turn, by the initial wetness of the concrete mixture. Excessively wet mixes are, indubitably, responsible for the majority of extreme cases of plastic flow. Delayed hardening, due to freezing or to low temperatures, and poor aggregates are responsible for most other cases.

The writer's attention was first called to the noticeable effects of plastic flow about seventeen years ago. Since then he has encountered several other extreme cases, but in practically every instance has been able to trace the condition back to one of the three causes mentioned.

Concerning the many statements found in engineering literature as to the inelastic properties of concrete, the writer, in his own experiments, has found that, generally speaking, for initial and repeated applications of stress, perfect elasticity is obtained in ordinarily good concrete up to 40% of the ultimate strength at the age tested. This 40% is a minimum value based upon concretes having a nominal 28-day strength of 2 000 lb. per sq. in., and more, and applies from ages as early as 7 days for concrete having a nominal 28-day strength of 3 000 lb. per sq. in., and more. For 2 000-lb. concrete, it has been found to apply at an age as early as 14 days.

---

[89] With Portland Cement Assoc., Chicago, Ill.

[90] *Proceedings*, Am. Soc. for Testing Materials (1929), p. 695.

[91] *Engineering and Contracting*, June, 1930, p. 234.

He has also found that linear relation of stress to strain up to a very high percentage of the ultimate strength may be obtained with the higher strength concretes on initial as well as on repeated applications of stress, the percentage decreasing with the strength of the concrete, but remaining well outside the range of the working stress for concretes having a nominal 28-day strength of 2 000 lb. per sq. in., and more.

Although 28 days is commonly accepted as a criterion in the adoption of working stress and elastic properties for design purposes, consideration should also be given to the increase of modulus with age and the coincident raising of the proportional limit during the process of hydration.

Arguing from these premises, it becomes questionable as to whether or not the author's attempt to provide for a variable stress-strain relationship in the design of concrete arches is of practical value.

A. Floris,[92] Esq. (by letter).—Although the use of masonry arches in structures is very old, the correct interpretation of their statical behavior is of a comparatively recent date. In the development of their theory the French and German engineers took a leading part.

The introduction of the elastic theory in the analysis of masonry arches, based upon Hooke's law as an approximation, was a marked advance in the design of arches. This theory satisfied the requirements of practice, as long as the application of concrete arches to long-span bridges was somewhat restricted. However, the use of arches with great spans is increasing steadily and a more elaborate analysis of their stress condition is highly desirable.

It is very gratifying, therefore, that the author, in his pre-eminently able analysis indicates further improvements in the design of concrete arches, which are of a distinct and practical value. Basing his thesis on the Bach-Schüle exponential law of elastic deformation and deriving from available experimental data an exponential law for the plastic flow of the concrete under sustained stress, Mr. Straub builds a new theory of arches which is a decidedly better approximation to the actual stress condition than the method founded on the linear distribution of stress, which disregards the time element.

It cannot be expected that empirical formulas such as those used in the paper will be perfect. They can be improved, however, as more information on the behavior of the concrete under stress accumulates. The remaining theoretical part of the author's analysis, being based upon the logical application of the laws of mechanics, is accurate and sound.

In the near future, perhaps, further progress in the stress analysis of concrete arches will be made, but the credit of first developing a more exact theory of concrete arches belongs rightly to the author. The paper deserves the closest attention of the students of concrete arches.

---

[92] Civ. Engr., Los Angeles, Calif.

Lorenz G. Straub,[98] **Jun. Am. Soc. C. E.** (by letter).—Much information and constructive criticism has been offered in the generous response of discussion by members of the Engineering Profession. However, despite the earnest effort to point out and distinguish between real and hypothetical considerations, there remained a certain amount of confusion in some instances.

There is a general expression of the viewpoint that much remains to be discovered regarding the time-stress-strain relations of concrete. Some of those who discussed the paper believe the separation of deformation into elastic and inelastic deformation is without justification.

Mr. Jakobsen calls attention to valuable information concerning the stress-strain relations for plain concrete. However, the writer does not feel that "the separation of elastic and plastic deformations seems an artificial procedure". It is true that this is a difficult segregation and that most "tests show the combined deformation only", but it is doubtful in the case of concrete whether "the plastic deformations disappear in due course of time, and are not in any sense permanent, except when the elastic limit has been exceeded". If so, then concrete must be considered to pass the elastic limit at a stress far below the normal working stress; in any case, experience indicates continuous deformation to occur in concrete when subjected to normal loading conditions. More experimental data would be very valuable in this connection.

As pointed out by Mr. Jakobsen if the stress-strain curve is linear for plastic as well as for elastic deformation, Hooke's law applies, and the time factor effects the deformations but not the load stresses. In this particular case, if no account were to be taken of the effect of time-yield in connection with temperature deformation, shrinkage, yielding supports, etc., the ordinary elastic theory would suffice completely for making a stress analysis of the structures; but even for the hypothetical case in which the stress-strain curve is linear for both plastic and elastic deformation, an analysis of the stresses incurred as a result of temperature changes, yielding supports, etc., requires that distinction be made between plastic deformation and elastic deformation.

The writer is in full accord with Mr. Jakobsen's statement that the exponential law for elastic deformation will not fully explain the somewhat perplexing problem of determining why plain concrete beams fail at such high computed stresses, although the computations based upon the law indicate a variation in the right direction. It is highly probable that more accurate information concerning the physical properties will shed more light upon this subject.

In connection with his statement that the writer "apparently computes the plastic deflection of a 140-ft. arch as 3 in., * * *," it seems that Mr. Turner has not clearly understood the diagram in question. Fig. 30 concerns a very hypothetical case in which the ratio, $\frac{H_{DL}}{H_{RS}}$, is considered equal to 25. Under such circumstances, the line of thrust will be eccentric with respect to its normal position. It was considered, therefore, that the line of thrust was not allowed to return to its normal position as a result of plastic flow, but instead was maintained eccentric in consequence of continuous contraction of the arch rib (temperature change), resulting in progressive angle changes within the arch rib. Obviously, this is merely a simple computation made for the purpose of showing that, for the particular hypothetical case, continued deformation might occur without change in the stress distribution in the arch rib. Computations for the case in which all deformation subsequent to removal of forms is due to plastic flow would indicate a crown deflection of only a fraction of the amount referred to by Mr. Turner, most of which would take place in the first few months. There is no question that a certain amount of rib-shortening and, consequently, deflection of the crown, occurs progressively at least during the early life of an arch. This is particularly emphasized by the result of rib-compensation methods (hydraulic jacks operated at the crowns of arches during the early life of the structures), used in France[94] and Germany. The results of experimental investigation[95] on arches by Wilbur M. Wilson, M. Am. Soc. C. E., also show continued deformation incurred under constant loading conditions.

---

[98] Associate Prof. of Hydraulics, Univ. of Minnesota, Minneapolis, Minn.

The writer is not in a position to say whether "Messrs. E. B. Smith, F. R. McMillan, and A. H. Fuller and C. C. Moore were mistaken in supposing they had discovered a general phenomenon of plastic flow in concrete as applied to the practical structure". Nevertheless, he does not doubt that they observed the phenomena of progressive deformation, and more recently that Raymond E. Davis, M. Am. Soc. C. E., and G. E. Troxell,[96] Assoc. M. Am. Soc. C. E., Mr. Oscar Faber,[97] and others observed that progressive deformation takes place in concrete over a period of many months under the influence of a continuously applied load. Mr. Turner's belief that a concrete arch having a cross-sectional area of 72 by 100 in. is very different from that of the ordinary test experiments, deserves careful consideration, because any such deviations are of great general importance in connection with experimental investigations of concrete.

Mr. Godfrey's timely emphasis of the psychological danger involved in the application of complex theories is worthy of special notice. Far too frequently fine-spun, precise analyses are worse than useless because of the false sense of security that they arouse in the design. Although he is no advocate of methodology in structural design, the writer desires to call attention to the noteworthy innovations in procedure for analyzing statically indeterminate structures which have recently been set forth by Hardy Cross, M. Am. Soc. C. E.[98] These should go a long way toward overcoming the mechanical methods of applying the theory of elasticity.

The concise, scholarly discussion by Professor Griffith considers many additional factors and does much to clarify the confused viewpoints in the interpretation of the phenomena of elasticity, plasticity, and viscosity. Professor Griffith's arguments make it seem reasonable that concrete should be called a "visco-elastic" material and that the occurrence of time-yield might possibly better be called viscous flow rather than plastic flow. It is only reasonable that technologists and scientists should meet on a common ground as far as definition is concerned and, therefore, the remarks contained in this discussion are especially important.

---

[94] M. Freyssinet, *Le Genie Civil*, Vol. 79, p. 126, 1921.

[95] *Bulletin No. 202*, Eng. Experimental Station, Univ. of Illinois, 1930, p. 96.

[96] *Proceedings*, Am. Soc. for Testing Materials, 1929, p. 696.

[97] *Paper No. 4645*, Inst. C. E., London,

[98] *Proceedings*, Am. Soc. C. E., May, 1930, Papers and Discussions, p. 919.

The writer does not consider the skepticism held by Mr. Comstock entirely justifiable. It is quite true that shear deformations result in plane sections not remaining exactly plane after flexural deformations occur. In the case of the arch, however, as pointed out in the paper (see "General Relations" under "Derivation of Proposed Theory of Elasticity for Arches"),

> "Since shear deformations have only slight effect on the change in conformation of a loaded arch rib in comparison to the effects of moment and rib-shortening deformations, the deformations due to shear will be neglected in the analysis."

It is fully agreed that in the case of pure flexures there will be a certain amount of approximation resulting from neglecting shear deformations, just as in the case of the usual approximate procedure of applying the ordinary theory of elasticity to beams. However, in the case of properly designed concrete arch ribs, the error introduced is unquestionably negligible.

Considering fixed-ended concrete arches, the writer cannot agree with Mr. Comstock's statement that "* * * it is of no importance whether or not the whole or any part of the strain disappears on removal of the applied forces". Where the deformation is progressive and dependent upon the concentration of stress, there results (in the case of arches and similar structures) a gradual displacement of the line of thrust which would not occur in the case of a perfectly elastic material. In this connection the writer wishes to emphasize the fact that, in order to make a satisfactory analysis where time-yield occurs, the entire structure must be taken into account and not merely its parts. In answer to Mr. Comstock's question why 1.3 was selected for the value of *m*, the writer wishes to refer to the paragraph following Table 3 in the paper:

> "It is realized, of course, from the experimental data on the elastic properties of concrete that a variable relation of stress to elastic strain in which *m* is as high as 1.3 is possibly never obtained in practice. However, an extreme value of *m* has been chosen in order that the results may show characteristic trends more prominently. Also, since a value of *p* of 1.3 in the proposed theory of plasticity is probably about correct for concrete 3 or 4 months old, this value makes a convenient starting point for an analysis in which the plastic properties of the material are considered."

The paper repeatedly emphasizes the need for more experimental data before a conclusive set of values can be chosen for the parameters in the proposed equations.

Furthermore, with reference to Mr. Comstock's statement that, "* * * it is not clear on what ground the author justifies the assumption that it [strain] may be expressed as the product of two functions, one of stress only and the other of time alone", the writer refers to Figs. 14 and 15. It will be observed that both these series of experiments indicate that the deformation is very well expressed by independent functions of stress and of time.

Unfortunately, not enough "time-yield" experiments extending over long periods are available at present, and the writer concurs in the opinion of Mr. Comstock that the value of m does vary with the age of the structure. Not only is this true, but the values of k, K, p, and q are also likely to vary with the age of the concrete, the latter even to a greater extent than m. No one questions that the modulus of elasticity varies with the age. However, the writer does not feel that this disproves the validity of the equations proposed. Extended experimental data may indicate—and the writer believes that this is not only possible but probable—that the parameters of the equations must be varied at intervals as the age of the structure increases in order to obtain satisfactory results, and it is possible that these variations might be quite large. In any case (although it is agreed that there is not sufficient experimental data to justify completely the general adoption of a stress-strain-time relation for concrete and that sometimes such studies may lead to preconceived notions), the writer feels that in not formulating reasonable relations based upon pioneer experimental studies an investigator is gathering information blindly without a definite purpose in view and, therefore, without recording all the facts desirable. This is true of much experimental data now available.

Considerable interesting and valuable information bearing upon the subject of this paper is brought to light by Mr. Eremin. Nevertheless, partial exception is taken to one of his statements referring to Fig. 21. If the properties of the material of construction are of a nature similar to those found from the experimental data available, there will be no appreciable redistribution of stresses in spite of the fact that excessive deflection occurs. It is true, however, that if the concrete at a particular point undergoes excessive yield similar to the condition which is likely to occur in the case of a steel beam, there would be a redistribution of stresses. Under such circumstances the effect would be similar to that of a settling support. However, although the center of a concrete beam may deflect progressively an excessive amount as a result of time-yield, this does not indicate a redistribution of stresses; on the contrary, the stresses are likely to remain practically unchanged.

The remarks by Dr. Melan and also those of Dr. Probst are of particular interest because of the broad experience of these men in connection with the design, construction, and experimental studies of arches in their respective countries. Dr. Melan's exception to the writer's conclusion regarding the early removal of forms is doubtless well founded and unquestionably such a procedure should not be adopted without careful precautions.

Dr. Vogt and Professor Gilkey rightfully emphasize the need of more experimental data before an acceptable method of analysis, such as proposed by the writer, can be adopted. It is well to call attention to the fact that the properties of concrete vary with the age. Consequently, as is evident from a comparison of data now available, the values of the parameters in the exponential equations, particularly the value of q, vary greatly with the age of the concrete. For a very green concrete the value of q is quite low, while for an older concrete, q is larger by several hundred per cent. Consequently, the writer does not recommend that a single value of q be adopted for concrete for a long period of time. Just what values should be used will depend upon information obtainable only by further experimental work. It is agreed that the writer probably made his analysis cover a longer period of time than should have been assumed for the experimental data available. These studies were intended only to indicate the trends rather than to present numerically accurate results.

The writer does not feel satisfied with Equation (117), presented by Dr. Vogt and Professor Gilkey, for use in studies of indeterminate structures such as concrete arches. It is true that this relation apparently indicates the approximate relation of stress to total strain, but the equation is not sufficient to prove the writer's "Second Conclusion" that,

"If there is an external disturbance * * *, such as the settlement of a support, the plastic flow causes a gradual redistribution of stresses such that the structure approaches (but never reaches) its original state of equilibrium (no settlement of support)".

The statement quoted requires the further verification of the fact that as a result of time-yield the line of thrust again approaches its original position. Equation (117) in itself does not show this.

Even if the value, *m*, is greater than *p*—a condition which is conceivable, but not probable—certain advantages are given by the early removal of the arch centering, although unquestionably certain disadvantages are also introduced. For example, if at the time the centering is removed there is a certain amount of displacement of the piers, a re-adjustment will take place tending to cause the arch to approach its original state of equilibrium (no movement of supports); on the other hand, if *m* is greater than *p*, the disadvantage incurred is that the line of thrust will be pulled a short distance farther away from the center line of the arch rib than if *m* is equal to or less than *p*.

Very interesting opinions and further data are given by Messrs. Bauman, Staehle, and Floris. It is to be hoped that more long-time experiments will be performed, in order to give a more accurate knowledge of many of the physical properties of concrete and their variation with the age of the material.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

TRANSACTIONS

Paper No. 1777

THE CHESAPEAKE AND DELAWARE CANAL

BY EARL I. BROWN,[1] M. AM. SOC. C. E.

WITH DISCUSSION BY F. T. CHAMBERS, M. AM. SOC. C. E.

SYNOPSIS

This paper is a record of the construction stages of the Chesapeake and Delaware Canal. The history of the project from 1764 is outlined, with a description of successive developments to 1927. The paper contains quantities, yardage, prices, total costs, and methods of procedure in connection with dredging operations. It also includes descriptions of several of the bridges that now cross the canal. The paper closes with a brief outline of hydraulic theory as applied to the project.

INTRODUCTION

The Chesapeake and Delaware Canal was constructed during the period, 1825-1829, by the Chesapeake and Delaware Canal Company, organized for that purpose under authority of special legislation enacted by the States of Maryland, Delaware, and Pennsylvania. Each of these States, as well as the Federal Government, subscribed for stock in the Company.

The location selected (after prolonged investigations, and abortive attempts at construction on other locations) was an east and west line connecting St. Georges Creek on the Delaware River about 40 miles below Philadelphia, Pa., with Back Creek on Chesapeake Bay, about 54 miles east of Baltimore, Md. The length of the canal was about 14 miles, and the summit level 17 ft. above mean low water in the Delaware River. As originally constructed, the locks were 22 ft. wide and 96 ft. long, with a depth of 8 ft. over miter-sills. Extensive improvements were made about 1850 to 1855, when new locks were constructed, 24 ft. wide and 220 ft. long, with a depth of 10 ft. over miter-sills. The original Company operated it as a toll canal for 90 years.

---

[1] Col., Corps of Engrs., U. S. A., U. S. Engr. Office, Philadelphia, Pa.

On August 13, 1919, the canal was taken over by the Federal Government for the purpose of replacing it with a new sea-level canal of somewhat larger dimensions. The enlargement was effected during the years, 1922-1927, and normal traffic was maintained practically uninterrupted during that period.

Reconstruction involved the excavation of about 16 000 000 cu. yd. of earth, the construction of four highway bridges and one railway bridge, the relocation of the Delaware end of the canal, the construction of tide dikes, jetties, and other works of a miscellaneous character.

The deepest cut was through the summit divide where the general land elevation is 80 to 85 ft. above sea level. Practically all excavation was successfully done by pipe-line dredges. Most of the spoil was deposited in specially prepared basins on the adjacent banks, involving lifts of 80 to 95 ft.

All new bridges are of the vertical lift type, electrically operated. They were designed for heavy military loads, and the foundations were built in contemplation of a future deepening of the canal to as much as 30 ft. The navigable span provides an opening of 175 ft. clear width, and a vertical clearance of 140 ft. above mean low water, when open.

The canal was opened to traffic at sea level on May 14, 1927, with approximately full project depth of 12 ft. at mean low water and a bottom width of 90 ft.

HISTORICAL

The two great bays, the Delaware and the Chesapeake, make deep indentations into the Eastern Coast of the United States. The northern end of Chesapeake Bay, inclining to the eastward, approaches close to Delaware Bay at a point where the latter, as the Delaware River, attains its farthest westerly meander; the distance between them at this point is only 20 miles. Both sides of the intervening peninsula are again deeply indented by tributaries which often approach to within a few thousand yards of each other at their head-waters. Chesapeake Bay, in this upper region, is marked by shores that are bluff, steep, and quite free from marshes; the outlets to its rivers are open and clean; the water is broad and fairly deep; the bottom is hard and usually sandy.

Delaware Bay has quite different characteristics, broad marshes border its shores with a width of 3 to 4 miles at places. The banks are so low that 2 to 3 ft. of water cover them at high tides; the outlets of its tributaries are obstructed by mud-bars. The high ground between the two bays is rather low in the broader parts of the peninsula farther to the southward, but at the canal it is 80 to 90 ft. above sea level.

The short distance at the upper end of the peninsula early invited plans for connecting the two bays by a navigable channel so that commerce would not be compelled to follow the sea route around, or make the portage across.

The second survey made for a canal in America is said to have been for connecting these bays near the present location. This was about 1764. Another survey followed in 1769, but the project was interrupted by the Revolutionary War. It was begun again in 1799, after the passage of an act by the Legislature of Maryland incorporating the Chesapeake and Delaware Canal Company. Similar acts were passed by Delaware and Pennsylvania and each State contributed toward the purchase of stock. The Federal Government likewise took an interest in the matter and subscribed for stock to the amount of $450 000. Pennsylvania contributed $100 000; Maryland, $50 000; and Delaware, $25 000. The cost estimated at that time was $2 250 000, and the remainder ($1 625 000) was to be obtained by popular subscription.

The first route selected (see Fig. 1) was an unfortunate one, and the plans (which provided for a canal with fourteen locks from Elk River to Christiana River) had to be abandoned for financial reasons as well as for lack of a suitable water supply. The City of Philadelphia opposed it because of an idea that it was unduly favorable to Wilmington, Del.

Actual construction operations had been begun in 1804, but were suspended after about two years. In the meantime additional studies were made, particularly from 1822 to 1824. These led to the definite abandonment of the old route and the selection of the one actually followed, which is substantially that still in use. The Company was re-organized and refinanced, so that it was able to begin active field work in 1824 and to complete the work in 1829. It was a large undertaking for the period.

Stung by the rapid growth of Philadelphia and New York, N. Y., the City of Baltimore began to take an interest in finding a shorter route to the sea and to northern ports. In 1871 a commercial convention was held in Baltimore and a movement toward establishing a ship canal between Chesapeake and Delaware Bays was inaugurated. It appeared that a saving of approximately 320 miles would be effected by such a canal. As a result the House of Representatives requested information of the Secretary of War. In 1882, the River and Harbor Act directed that a survey be made of various routes. Several District Officers and Boards reported on possible alternate routes, but all finally came to the same conclusion, that the existing canal route was the most feasible.

As a net result the United States finally decided to buy the canal and to re-construct it so as to eliminate locks and increase its capacity. The immediate reason was to use the canal as a link in a proposed intra-coastal system along the Atlantic Coast. However, it has been generally agreed that the time would probably soon come when a ship canal along this route will be a necessity. This thought has guided all the working plans for the reconstructed canal.

DESCRIPTION

The canal was acquired by the United States on August 13, 1919, for the purpose of constructing along its route a sea-level barge canal, with a depth of 12 ft. at mean low water for a bottom width of 90 ft.

The original canal had its entrance at Delaware City, Del., about 40 miles south of Philadelphia (see Fig. 1). Here a tide-lock maintained a canal level of 7.6 ft. above mean low water in the Delaware River. Thence, the canal extended between banks, partly natural and partly artificial, for a distance of 4.3 miles to the Town of St. Georges, Del., its level at some of the intermediate points being above that of adjacent marshes. Because the banks become higher near St. Georges the canal level there was raised 10 ft. by

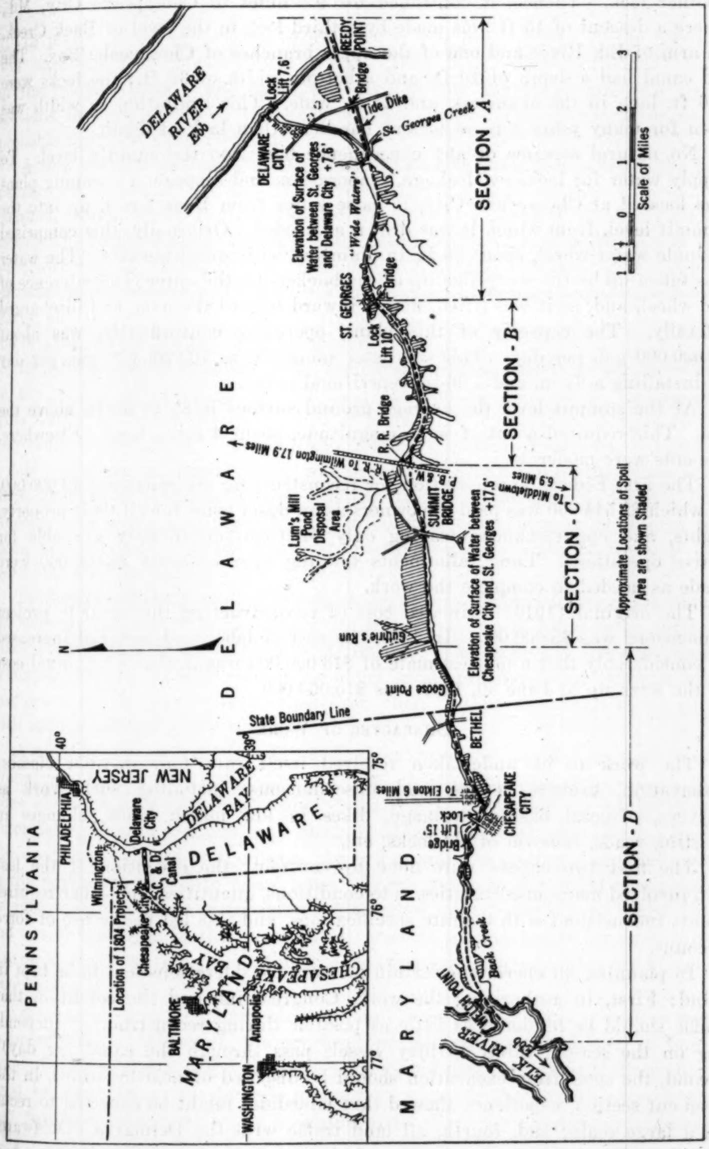

PENNSYLVANIA
NEW JERSEY
PHILADELPHIA
Wilmington
Delaware City
C. & D. Canal
Location of 1804 Project
Chesapeake City
BALTIMORE
Annapolis
WASHINGTON
MARYLAND
DELAWARE
DELAWARE BAY
CHESAPEAKE BAY
40°
39°
75°
76°
77°
N
DELAWARE RIVER
Ebb
DELAWARE CITY
Lock Lift 7.6'
REEDY POINT
Bridge
Tide Dike
St. Georges Creek
Elevation of Surface of Water between St. Georges and Delaware City +7.6'
"Wide Waters"
ST. GEORGES
Lock Lift 10'
Bridge
SECTION - A
SECTION - B
R.R. Bridge
R.R. To Wilmington 17.9 Miles
P.B. & W.
To Middletown 6.9 Miles
Lum's Mill Pond Disposal Area
SUMMIT BRIDGE
SECTION - C
Guthrie's Run
Goose Point
Elevation of Surface of Water between Chesapeake City and St. Georges +17.6
Approximate Locations of Spoil Area are shown Shaded
State Boundary Line
BETHEL
To Elkton 6 Miles
CHESAPEAKE CITY
Lock Lift 15'
Bridge
SECTION - D
Back Creek
WELCH POINT
ELK RIVER
Ebb
D E L A W A R E
M A R Y L A N D
0 1 2 3
Scale of Miles

FIG. 1.—CHESAPEAKE AND DELAWARE CANAL: LOCALITY SKETCH.

another lock. Thence, it continued for 9.3 miles to Chesapeake City, Md., where a descent of 15 ft. was made by a third lock to the level of Back Creek, an arm of Elk River and one of the upper branches of Chesapeake Bay. The old canal had a depth of 10 ft. and a bottom width of 36 ft.; the locks were 220 ft. long in the chambers, and 24 ft. wide. This limitation in width was even for many years a more serious handicap than lack of depth.

No natural streams of any consequence flow into the summit level. To supply water for lockages, leakage, evaporation, and seepage, a pumping plant was located at Chesapeake City, to raise water from Back Creek up into the summit level, from which it was drawn as needed. Originally, this comprised a single water-wheel, about 35 ft. in diameter, with spiral buckets. The water was taken up by the scoop-like lip of the buckets on the outer circumference of the wheel, and, as it was lifted, flowed inward toward the axis, and discharged laterally. The capacity of this plant, operating continuously, was about 40 000 000 gal. per day. This was later increased to 100 000 000 gal. per day by installing a 24-in. and a 36-in. centrifugal pump.

At the summit level the average ground surface is 85 to 90 ft. above the sea. This required a cut of some magnitude, about 4 miles long. Elsewhere, the cuts were moderate.

The first Federal appropriation for reconstructing the canal was $3 000 000, of which $2 514 000 was paid the owners as an upset price for all their property, rights, and appurtenances, leaving only $480 000 immediately available for active operations. Later, allotments totaling approximately $8 000 000 were made as needed to complete the work.

The original (1916) estimated cost of reconstructing the canal to project dimensions was $8 000 000. In 1919, the cost of labor and material increased so considerably that a new estimate of $13 000 000 was made. The actual cost of the work up to June 30, 1927, was $10 060 000.

Character of Work

The work to be undertaken resolved itself into four distinct classes: Excavation, bridges, jetties, and miscellaneous, including such work as surveys, disposal basins, drainage, dikes in low marsh lands, changes in existing roads, removal of old locks, etc.

The first two classes were done by contract; the execution of the last two involved many uncertainties as to conditions, quantities, and other requirements inconsistent with definite specifications, and this led to the use of force account.

In planning all operations certain primary considerations had to be kept in mind: First, in authorizing the work, Congress imposed the condition that traffic should be hindered as little as possible during reconstruction (depending on the season, thirty to fifty vessels pass through the canal per day); second, the spoil from excavation should be disposed of near-by; third, in the deep cut section, experience showed that landslides might be expected to recur on a large scale; and, fourth, all land traffic with the Delmarva (Delaware-Maryland-Virginia) Peninsula crossed the canal on the various bridges, and these communications could not be broken.

PLANNING THE WORK

In planning the excavating operations, it was necessary to decide between wet excavation or steam-shovel work. This involved an extended study of the relative costs. The canal was divided into four approximately equal sections in each of which the work was of about the same character as to probable unit cost of excavation.

A new eastern entrance to the canal was to be provided, because the old passage at Delaware City was defective in that it debouched into a secondary channel, which, while deep enough for the present project of 12 ft., would be insufficient and would involve a greatly increased volume of work should the waterway ever be enlarged to a ship canal. A second reason is that the new entrance is placed south of Fort du Pont, one of the defenses of the Delaware River, and this adds to the military strength of that point. The new line joins the original canal about two miles inland.

In the light of all available facts, dredging (without first lowering the water in the canal) appeared to be the most satisfactory method of excavation. This would minimize the occurrence of slides. By cutting the bottom to its new proposed elevation (— 12 ft.) and holding the summit level at + 17.6 ft., a depth of water of almost 30 ft. was provided. Slides might then flow with little danger of obstructing the existing traffic. The use of dredges was objectionable because of the difficulty of finding suitable near-by dumps for spoil, and of getting a sufficiently powerful dredging machine through the locks and into the canal.

To dispose of the material conveniently, it was decided to purchase about 300 additional acres of land adjacent to the canal, and to build large earthen dikes around this tract to form spoil basins. The high lift for the pumps (ranging from 60 to 90 ft. above the canal level) precluded any but powerful machines, and it was believed that even then "boosters" would be required with each machine.

A method of introducing large dredges into the canal was devised at Chesapeake City. Two coffer-dams of steel sheet-piling were provided in two sides of the saving basin[2] adjacent to the canal. By removing one of these coffer-dams, communication was established at tide level with Back Creek. The dredge was placed in the basin, the coffer-dam was restored, and the water in the basin was raised to canal level. Then, access to the canal was gained by removing the other coffer-dam. Altogether, six dredges were taken into and out of the canal in this manner. In making arrangements for excavation contracts, small sections, including the existing and proposed sites for bridges, were excluded.

One railway, one secondary highway, and three main highway, bridges were to be provided. Although the sea-level project contemplated only a barge canal of limited depth and width, it was conceded that such limited dimensions must be regarded as a relatively temporary stage in the canal's growth and that future enlargement would be required to allow the passage of large sea-going vessels. Besides this idea of a larger canal, there were also traffic and foundation questions that led to the adoption of dimensions and clearances for all bridges such as would conform with a future depth as great as 35 ft. The bridges were intended to carry the heaviest military loads. Based upon these considerations, the characteristics imposed on design were: Depth of foundations of main piers, at least 40 ft. below mean low water; minimum horizontal clearance of navigable span, 175 ft.; minimum overhead clearance for fixed or vertical lift bridges, 140 ft.; floors with bridges closed, approximately at present road levels; and all bridges power-operated.

---

[2] A saving basin is a basin provided alongside a lock into which the upper half of the prism of water between upper and lower levels may be drawn in emptying the full lock, and there preserved for use in filling the lower half when the lock is next filled. The net loss of water in a lockage is therefore only one-half, instead of an entire, lock full.

SURVEYS AND PLANS

Surveying for the construction of the sea-level canal was begun immediately after the United States took the project from its former owners on August 13, 1919. The alignment adopted was that laid down by the Board of Engineer Officers convened in compliance with the River and Harbor Act approved March 3, 1909, and charged with making survey and reports for a continuous waterway from Boston, Mass., to Beaufort, N. C.[3] The task of preparing the requisite working plans and maps included three major surveys: (1) A topographical survey of the entire canal from Delaware River to Elk River, with all adjacent lands for studying the spoil-area problem, and for general information; (2) location of all property line corners to determine whether or not the existing rights of way were of sufficient width, throughout, to accommodate the sea-level canal; and (3) laying out the new canal on the ground, with cross-sections at 100-ft. stations along the center line, for detail work.

Stadia and plane-table methods were used in the topographical surveys and contours were located at 10-ft. intervals. The area covered was about 50 sq. miles and the average width was 2.5 miles. The survey was platted to a scale of 1:9 600, thus requiring three standard 27 by 40-in. sheets for the entire route.

In the next step all property acquired for the new canal was staked out and permanent monuments were placed at controlling and meander corners. The property lines, thus located, were platted on the working drawings to a scale of 1: 1 200 so as to show the position of the new canal with respect to the right of way.

The general line of the sea-level canal is substantially the same as the old one, except at the east end, where it strikes the Delaware River about 1¾ miles below the Delaware City lock.

As a part of the layout work, range points for tangents and chords and points of intersection of tangents were monumented. Cross-section profiles were made at each 100-ft. station. Elevations were taken every 10 ft. horizontally at right angles to the center line for a distance of 500 to 800 ft. on each side. Soundings and wye-level measurements were made from the 12-ft. depth in the Delaware River to the Pennsylvania Railroad Bridge, the eastern end of "Deep Cut" Section, and from Guthries Run, the western end of "Deep Cut" Section, to Elk River (see Fig. 1). In "Deep Cut" where the adjacent hills were from 20 to 100 ft. above the canal level, this method was supplemented by hand-leveling, trigonometric leveling, and checking on a wye-level line at the top of the slope along the steep banks.

---

[3] H. R. Doc. 391, 62d Cong., 2d Session.

These data were reduced and the plan was platted to a scale of 1:1 200 which covered 21 standard size sheets 27 by 40 in. The cross-sections were platted to a scale 1 in. equals 100 ft. horizontal and 20 ft. vertical. These drawings were the basis of all the necessary designing and computations for the contract specifications and for hired labor work.

Excavation

The canal was divided for excavation purposes into four sections. (See Sections A, B, C, and D, Fig. 1.) Practically all the material to be removed in Section A was under water; hence, it was a dredging proposition. In passing the dredge from the new sea-level entrance to the intermediate level it was important not to permit the water in the embanked part of the old canal to escape. This was prevented by using excavated material to build a dam or dike across the dredged channel, thus leaving the dredge floating in a completely enclosed pond. Water from the canal was then admitted into this pond raising the dredge to canal level, which was retained by the dam. The dredge was then ready to continue on its way into the old canal and complete its work. The operation was reversed in coming out.

Section B contained practically no material above the canal surface, and it was clearly a dredging proposition. The only question to be decided was as to whether a dredge could be introduced into the canal through the locks or by other means.

In Section C (Fig. 1) the height of the banks averaged 50 to 75 ft. above the water of the summit level (17.6 ft.) The excavation was approximately 50% of the total required to complete the 12-ft. canal.

This section of the canal was called the "Deep Cut". It was here that the choice between wet and dry excavation was most difficult to make. The original builders of the canal had piled material from this cut close along its edges, thereby greatly overloading the banks and adding considerably to the material now to be removed for the enlargement. Approximately, 7 000 000 cu. yd. were to be taken from this cut, of which perhaps two-thirds could be removed by dry excavation after first deepening the summit level enough to lower it 10 ft. Even then the remainder below water level would have had to be dredged, and slides would very probably have been induced by this preliminary lowering. The occurrence of such slides in so shallow a channel would have practically closed it to all traffic. Section D was clearly a case for dredging. The combined length of all sections was 20 miles, and the total quantity to be excavated was 16 000 000 cu. yd.

Contracts for Excavation.—The first shovelful of dirt was raised under the new project June 27, 1921, when a preliminary trial contract for steam-shovel excavation was awarded for removing the overburden down to Elevation + 20 (see Table 1, Item 1). The material was transported by three trains of dump-cars to the disposal grounds in Lums Mill Pond which, previous to the acquisition of the canal by the Federal Government, had been used as a feeder

TABLE 1.—DATA RELATING TO QUANTITIES AND COST OF EXCAVATION.

Item No.	Section.	Date operations began.	Date operations ended.	Size of hydraulic dredge intake, in inches.	MATERIAL REMOVED, IN CUBIC YARDS.			Cost, in dollars, paid to contractor.	Contract unit price, in dollars per cubic yard.	Remarks.
					Gross.	Excess.	Net.			
					PRELIMINARY TRIAL CONTRACT BY STEAM SHOVEL.					
1	C	6-27-21	3-30-23	....	650 924	....	650 924	$ 240 841.88	$0.37	Between Pennsylvania R. R. Bridge and Summit Bridge.
					CONTRACTS FOR IMPROVEMENT.					
2 3 4	A	10-15-23	12-30-24	20 23 22	2 419 486	698 184	1 721 302	$ 209 905.22	$0.1244	Three dredges working at different times.
5	A	8- 3-26	3-22-27	22	87 927	31 522	56 405	21 180.80	0.385	
6					2 507 413	729 706	1 777 707	$ 231 086.02		Total for Section A.
7	B	10- 1-23	12- 3-24	22	2 185 076	375 183	1 809 943	$ 247 448.07	$0.139	
8	B	8- 3-26	3-22-27	22	113 086	10 379	102 707	39 027.70	0.385	
9					2 298 162	385 512	1 912 650	$ 286 475.77		Total for Section B.
10 11	C	12-10-22 12-11-23	12-29-24 12-29-24	26 26	5 515 394	59 298	5 456 096	$1 334 230.00	$0.2453	Between Guthries Run and Summit Bridge.
12 13	C	7-21-24 8-25-24	11-10-24 11-15-24	22 26	1 276 952	74 459	1 202 493	274 683.23	0.23	Eastern end of section between Pennsylvania R. R. Bridge and Summit Bridge.
14	C	3- 6-25	5-12-25	26	495 435	48 933	451 502	193 417.01	0.43	
15					7 287 781	177 690	7 110 091	$1 802 280.24		Total for Section C from 1922 to 1925.
16	C	8- 3-26	3-22-27	22	248 569	12 599	235 970	90 630.77	0.385	
17	C	8- -24	11- -24		7 536 350	....	7 346 061	$1 892 911.01		
18				Shovel	56 000	....	56 000	42 000.00	0.75	
19	C				7 592 850	190 289	7 402 061	$1 934 911.01		Total for all of Section C.

TABLE 1.—(*Continued*).

Item No.	Section.	Date operations began.	Date operations ended.	Size of hydraulic dredge intake, in inches.	Material Removed, in Cubic Yards.			Cost, in dollars, paid to contractor.	Contract unit price, in dollars per cubic yard.	Remarks.
					Gross.	Excess.	Net.			
					Contracts for Improvement.—(*Continued*).					
20	D	9-24-23	12-31-23	20	2 858 864	592 062	2 266 802	$ 537 243.13	$0.2447	Elk River Channel.
21		10-18-23	5-17-24	22						
22		12-12-23	6-11-24	27						
23	D	1- 8-25	5-17-25	26	300 070	51 927	248 143	105 401.82	0.43	Small stretch west of locks at Chesapeake City.
24					3 158 934	643 989	2 514 945	$ 642 644.95		
25	D	8- 3-26	3-22-27	22	50 087	5 450	44 637	16 660.05	0.385	
26					3 209 021	649 439	2 559 582	$ 659 305.00		Total for Section D.
27					16 257 870	1 954 946	14 302 924	$8 352 619.68		Total for all improvement contracts.
					Contracts for Maintenance Work.					
28	A	1926	1927	22	903 792	401 127	502 665	$ 154 997.04	$0.334	Two electric dredges used on this work.
29	B	....	....	....	....	....	....	....	......	
30	C	1926	1927	22	573 169	184 088	389 081	121 270.73	0.334	
31	D	1926	1927	22	565 747	177 083	388 664	124 820.31	0.334	
32		8- 3-26	3-22-27		2 042 708	762 298	1 280 410	$ 401 088.08		Total contracts for maintenance.
					Improvement Dredging with Leased Equipment.					
33	B	1926	1927	Scoop	9 925	....	9 925	$ 7 262.50		
34	C	1926	1927	Scoop	4 725	....	4 725	16 650.00		
35	D	1926	1927	15	31 157	....	31 157	36 442.33		
36					45 807	....	45 807	$ 60 354.83		Total improvement dredging by leased equipment.

TABLE 1.—(*Continued*).

Item No.	Section.	Date operations began.	Date operations ended.	Size of hydraulic dredge intake, in inches.	MATERIAL REMOVED, IN CUBIC YARDS. Gross.	Excess.	Net.	Cost, in dollars, paid to contractor.	Contract unit price, in dollars per cubic yard.	Remarks.
MAINTENANCE DREDGING WITH LEASED EQUIPMENT.										
37	B	5- 1-27	5- 8-27	Scoop	545	....	545	$ 1 500.00		
38	B	2-18-26	7- 2-26	15	216 889	....	216 889	65 347.92		
39	C	3-22-27	6-22-27	22	196 382	....	196 332	113 119.80		
40	D	....	....	....	....	....	....	....		
41					413 766	....	413 766	$ 179 967.72		Total maintenance dredging by leased equipment.
IMPROVEMENT EXCAVATION BY GOVERNMENT EQUIPMENT.										
42	A	7- -24	5- 4-26	Shovel	15 000	....	15 000	$ 28 778.27		Removal of old lock at St. Georges.
43	B	....	....	....	....	....	....	....		
44	C	10- 7-25	2-17-26	Shovel	60 371	....	60 371	39 954.40		Summit Bridge.
45	D	3- -24	10- 5-25	Shovel	49 532	....	49 532	45 491.05		Old lock at Chesapeake City.
					124 903	....	124 903	$ 114 223.72		
SUMMARY OF TOTAL DREDGING.										
46	A	10-15-23	1927	....	3 426 205	1 130 833	2 295 372	$ 414 861.33		
47	B	10- 1-23	1927	....	2 525 521	385 512	2 140 009	360 586.19		
48	C	6-27-21	1927	....	9 077 871	374 377	8 703 494	2 466 747.82		
49	D	9-24-23	1927	....	3 855 457	826 522	3 028 935	866 058.69		
50					18 885 054	2 717 244	16 167 810	$4 108 254.08		Grand total for all dredging.

reservoir to supply water to the summit level. The retaining earth dam had been washed out by a freshet August 14, 1919, immediately after the Government had taken over the canal, and the spoil of the operation was used to good advantage in rebuilding the dam, which thus provided a spoil area for material dredged under later contracts.

After this preliminary trial of steam-shovel excavation, dredging was chosen for the remainder of the work. The "Deep Cut" was recognized as the key to the entire job. Therefore, specifications were prepared, after an allotment of $2 500 000 had been made, with a view to placing a contract for that part of Section C comprising the most difficult work between Guthrie's Run and Summit Bridge. About 5 000 000 cu. yd. of earth were involved in this contract. The specifications were made so as to permit the contractor to use either wet or dry excavation with any combination of methods he might choose.

Operations were begun on December 10, 1922, with a 26-in. hydraulic dredge which had previously entered the canal through the coffer-dam at Chesapeake City. At the same time another 26-in. dredge was brought in; but it was still under construction and did not start operating until one year later, December 11, 1923. (See Table 1, Item 11.) Previous to the beginning of dredging operations a force of men with tractors, dynamite, etc., were at work clearing the north bank of trees, stumps, underbrush, and roots, and constructing a retaining bank and waste weir for the Guthries Run spoil area (Fig. 1). The first retaining banks and the waste weir were soon found to be inadequate because of the great quantity of water being pumped with the spoil. This made a more substantial construction necessary.

The method then chosen by the contractor to carry out this work was, briefly, as follows: A steam shovel and the dump-cars from the former contract were used to remove the top of the spoil bank of the old canal. With this material, a heavy earthen embankment varying in height up to 25 ft. was built parallel to the canal and not less than 300 ft. from it, to form one side of the basin into which the spoil was to be placed; usually the other sides were thrown up by a drag-line excavator. As the dredge had an average daily output of about 7 000 cu. yd. on this work, cross-dikes were run, as needed, to provide smaller basins into which it could discharge without interfering with the construction of the main dike farther along. The contractor purchased about 500 acres of land for this purpose which, added to approximately 300 acres owned by the United States, made a total of about 800 acres devoted to these spoil basins. The embankment on the canal side was about 24 000 ft. long. The waste water from the entire area was forced by a series of levees, weirs, and ditches to settling ponds north of a public road paralleling the canal. After a travel of 2 to 3 miles through these basins, the water returned to the canal quite clear. This return was quite essential because all the water in the canal had to be supplied by the Chesapeake City pumping plant, the capacity of which would not have been adequate otherwise.

The material encountered in this section consisted of strata of loam, sand, some gravel, hard blue mud, and clay. The hard blue mud and the clay could not be broken up, but came through the pipe line in slices and lumps, piling up at the end of the pipe line without running off.

There was little delay caused from choking of the pipe line, although the lines were run up almost perpendicular banks from 60 to 80 ft. high. The greatest height over which material was discharged was 97.5 ft. above the water level in the canal. The major part of the rip-rap stone revetment of the old canal along the old tow-path was removed by a small bucket dredge, before the dredging began.

The only damage done to plant in boldly attacking banks of maximum height (aside from normal deterioration) was an occasional broken spud or hoisting cable. The character of material was such that instead of flowing to the intake pipe of the dredge under a gradual slope, large masses of it would either slide or break off and fall into the water without warning; this caused waves which would occasionally break the spuds before they could be hoisted. No structural damage was done to the plant. The difficulty was partly overcome by washing the banks down by flowing water or jets; but the most successful method was by the use of blasting powder, first drilling a series of holes along the required slope line to depths varying from 10 to 25 ft. and about 20 ft. apart. The explosion of this powder would usually cause the earth to fall into the head of the cutting in a more gradual manner.

The part of Section C from the Pennsylvania Railroad Bridge to Summit Bridge was awarded in two separate contracts. On July 21, 1924, a 22-in. hydraulic dredge started at a bend in the old canal 1 600 ft. west of the Pennsylvania Railroad Bridge and worked east, leaving the old canal to the north (Fig. 1). Since the only available spoil area was Lums Mill Pond, it was necessary to lay a submerged pipe line across the old canal to dispose of the dredged material without interrupting traffic. On August 25, 1924, another dredge started at the same bend and worked west, spoiling in the same area through a pipe line from 1 000 to 4 000 ft. long, with the discharge at Elevation + 42.5 ft. The total quantities excavated on this contract are given in Table 1 (Items 12 and 13).

Operations were begun on a new contract on March 6, 1925, by a dredge, working west from a point 1 200 ft. east of Summit Bridge (Fig. 1) where the overburden had previously been removed to Elevation + 20 by the steam shovel. An area just east of the public road leading from Summit Bridge to Glasgow, Del., purchased for spoiling purposes, was enclosed by 6 000 ft. of levee. (See Table 1, Item 14.)

Section B is about 3 miles in length, extending from St. Georges to the Pennsylvania Railroad Bridge. A 22-in. hydraulic dredge entered the canal through the coffer-dam at Chesapeake City, proceeded east to St. Georges, and began operations on October 1, 1923. (Table 1, Item 7.) The course of the old canal was followed westward from St. Georges to Lorewood Grove. At this point the tow-path was cut through and the new canal diverged gradually to the north to eliminate bends. The greatest divergence was approximately 300 ft. passing through Otter Island and a point of high land with an elevation of + 37 ft. on the north, then crossing the old canal to the south of the

Pennsylvania Railroad Bridge. This provided an improved approach to the new bridge to be constructed for the Pennsylvania Railroad.

This course necessitated the clearing of some woodland by removing the trees and stumps with dynamite and tractors, and grubbing the underbrush and roots ahead of the dredge. The dredged material consisted of sand, clay, and gravel and was deposited with little difficulty in the coves and behind the old tow-path. A small dipper dredge was used to remove the rip-rap stone from the old tow-path in the line of the new canal and to construct approximately 2 500 ft. of retaining bank.

Dredging operations on Section A began at Reedy Point, Del., with a 23-in. hydraulic dredge, working westward through the low marshland and the upland to the Delaware City-Port Penn Road. At this point operations were suspended on December 16, 1924, to prevent the flooding of several hundred acres of low land lying to the south. After resuming dredging operations at the Delaware City-Port Penn Road on July 7, 1925, the contractor's dredge continued westward to join the old canal at "Wide Waters." The mud, sand, and gravel dredged up to the road was deposited in spoil areas on the north side of the canal. Two of these were low, swampy places on the Fort du Pont Reservation, which required the construction of approximately 3 800 ft. of pipe line. The only difficulty encountered was caused by the sand that settled behind stones lodged in the line. The bulk of material dredged from the canal prism in this stretch was deposited along the face of the dike as a reinforcement and protection against erosion from tides and wind.

Before a dredge could enter the old canal at "Wide Waters" it was necessary to construct a levee behind it, 600 ft. long, with the top at Elevation +14.0, from the high land on the south to the old canal bank on the north. The water from the canal was let into the basin, thus raising the dredge to canal level, at which it then continued operations westward to St. Georges. The material dredged through "Wide Waters" was principally vegetable matter, which would rise to the surface of the water in lumps and float off like small islands. A mixture of mud, sand, clay, and gravel that impeded progress was encountered near St. Georges. The material was deposited in the coves along the south side of the canal and on the low lands to the north.

Three hydraulic dredges were in operation on this contract at different times; one of them did 45% of the work and completed the contract on December 30, 1924. It passed out of the canal the same way that it entered. The quantities dredged under this contract are listed in Table 1 (Items 2, 3, and 4).

Section D was awarded in two separate contracts (see Table 1, Items 18, 19, 20 and 21). That of the Atlantic, Gulf, and Pacific Company covered the whole section except a small stretch from a point just west of the locks at Chesapeake City. A total of 830 700 cu. yd. of mud, sand, clay, gravel, and some sandstone was excavated from the 4¼ miles of channel from Elk River to Chesapeake City. It was deposited in the coves on both sides of the channel without difficulty other than that usually attending dredging operations. The channel was completed to Chesapeake City on December 31, 1923.

Two dredges were admitted into the canal separately through the cofferdam at Chesapeake City and worked east along the lines of the old canal to

Guthries Run, a distance of 2½ miles. From a point 600 ft. east of the old canal waste-gates at Chesapeake City to the Maryland and Delaware line the material consisted of a tough red clay with a small percentage of sand and gravel, which was easily disposed of in the coves to the south and behind the old tow-path on the north side. From the Maryland and Delaware line east to Guthries Run the land rises from the canal level to Elevation +45. The contractors purchased two tracts of land for disposal areas, one on each side of the canal. To enclose these basins required approximately 12 000 ft. of levees which were constructed with dredged material in a series of steps from canal level to Elevation +50. The material was mostly sand, loam, and gravel, with a small percentage of red clay. Considerable difficulty was encountered in maintaining the levees, and several washouts that released thousands of cubic yards of sand, made re-dredging necessary; but this did not impede traffic in the canal to any great extent.

On January 8, 1925, a 26-in. dredge began work 600 ft. east of the waste-gates, which work was extended west to the old drawbridge at Chesapeake City, a distance of 1 800 ft. On February 28, 1925, work was suspended and the dredge was moved to Section C until May 12. Work was resumed on May 13, and the contract was completed May 17, 1925. The material, consisting of red clay and sand, was deposited in Broad Creek spoil area.

There were eight separate contracts for excavation awarded from June, 1921, to March, 1925, four to W. H. Gahagan, Incorporated, two to the Arundel Corporation, and two to the Atlantic, Gulf and Pacific Company. The plant in operation consisted of one steam shovel, seven powerful hydraulic dredges, two bucket dredges, two scoop dredges, two dragline banking machines, and a number of small gasoline tugboats used for transporting men and towing fuel-oil barges, etc., to and from the various plants. Each dredging company proved its efficiency to cope with such a large undertaking in the smooth and satisfactory manner in which every obstacle encountered was met and they completed all contracts in advance of the specified time. The total quantities removed from the canal prism under these contracts were 16 202 000 cu. yd. of material at a cost of $3 311 000, or, approximately, 20 cents per cu. yd.

Excavation work was suspended after May 17, 1925, to await the completion of the highway bridges. Owing to a delay in the construction program there were four bridge sites at which no excavation was done under contract: St. Georges, 700 ft.; Pennsylvania Railroad Bridge, 500 ft.; Summit Bridge, 500 ft.; and Chesapeake City, 1 200 ft.; or, a total of 2 900 ft. of channel.

Only one man lost his life on all excavation contracts. He slipped from the pontoon line of one of the dredges and was drowned. The damage to plant other than broken spuds and the usual wear and tear attending dredging operations was slight. The regular boat traffic through the canal was not interrupted to any great extent except when the dredges were being taken in and out of the canal through the coffer-dam at Chesapeake City, which required two or three days for each operation.

In the meantime, Government plant began the removal of the old original locks at St. Georges and Chesapeake City, constructed between 1824 and 1829. A dam of timber and steel sheet-piling was first constructed at Chesapeake City to retain the canal level so that the lock could be kept in operation and yet all dredging of the channel around it could be completed except at the dam.

Leased Plant.—By January, 1926, sufficient progress had been made on the bridges and the removal of the old locks to resume dredging operations. A 15-in. hydraulic dredge and its attending plant was leased at $700 per day. It was the only hydraulic dredge available that could pass through the canal locks. Accordingly, it began work at Chesapeake City, where the old locks had been located, and dredged to project dimensions (Fig. 1). It was then moved to Summit Bridge, but could not pump the material to the elevation of the spoil area. Next, it was moved to the Pennsylvania Railroad Bridge and worked from that point to St. Georges, doing maintenance dredging until June 30, when it was released. It removed about 34 700 cu. yd. of mud, sand, gravel, and clay, under improvement and 217 000 cu. yd. under maintenance, at a total cost of $106 000.

When it was found that the 15-in. dredge could not complete the reconstruction of the canal and that a more powerful one would be required, negotiations were started for a 26-in. dredge to be leased for 60 days for $169 000 and at $1 500 per day thereafter. The dredge was available immediately and was started from New York City on March 30, 1926. It foundered en route off the New Jersey Coast during a storm on March 31, and was a total loss.

Final Contract.—After various unavoidable occurrences had impeded the progress of the work, plans and specifications were prepared to complete the work by contract. Bids were invited and the contract was awarded to the Atlantic, Gulf and Pacific Company, at 38.5 cents per cu. yd. for improvement and 33.4 cents per cu. yd. for maintenance dredging.

It was necessary to construct a coffer-dam at St. Georges to make a basin 190 ft. long and 140 ft. wide to raise the dredge to the upper canal level (Elevation +17.6) without interfering with traffic in the canal. Work was started about July 15, 1926, and was completed August 24. The old lock wall was used as the south side of the coffer-dam. The east side consisted of two rows of 35-ft. steel sheet-piling 15 ft. apart. These were tied together on the inside by cross-rows of steel piles spaced 20 ft. apart, thus forming pockets that were filled with earth. An opening for the dredge to enter was provided in the east end of the coffer-dam.

The north side was of the same construction with the two rows of piles, 14 ft. apart and from 14 to 35 ft. long, extending for a distance of about 70 ft. to high banks. Then a dirt bank, with wooden piles and sheathing as a core, extended up the side of the bank to Elevation +20.0. The west end of the coffer-dam was the old roadbed.

A 20-in. hydraulic dredge began operations on August 3, 1926, pumping a dam across the new entrance to the canal 15 200 ft. from Delaware River. This formed a basin by which to raise the dredge from tide level to canal level at Elevation +7.6. The dredge then cut through a dam constructed under a previous contract, entered the canal at "Wide Waters", and continued westward on maintenance dredging to St. Georges, entering the coffer-dam on August 25.

The time required to pass the dredge through the coffer-dam to the upper level was five days. Leaving St. Georges, it proceeded west to the Pennsylvania Railroad Bridge and began operations on improvement dredging at that point. It was engaged there until September 27, when it was found that the bridge piers were settling before project dimensions had been obtained in the draw-opening. The dredge was withdrawn and placed on maintenance work between the railroad bridge and Summit Bridge until October 11, while steps were taken to restore the bridge to condition. From October 12 to November 13, 1926, improvement dredging was completed at Summit Bridge, and the dredge continued westward on maintenance work until January 24, 1927.

Canal Lowered to Sea Level.—On January 25, 1927, the dredge was moved to Chesapeake City to dredge up to the retaining dam in preparation for lowering the water in the canal to sea level. The wickets were opened in the locks at Chesapeake City and Delaware City, on February 1, 1927, and, as the make-up water escaped, the dredge continued to work through the old roadbed. It reached the retaining dam on February 7, and was then moved back to complete the maintenance dredging between Chesapeake City and Bethel while a force of men removed the retaining dam. This was practically completed by February 20; dredging was resumed at that point on February 22 and completed on February 28, except a part of an old lock wall that could not be removed by the hydraulic dredge.

When the canal surface reached sea level a great number of small slides occurred along the banks of the canal, which usually did little or no damage to the channel. At a point 3 500 ft. west of Summit Bridge a slide occurred on the north side about 600 ft. long. It moved gradually into the channel, forming a shoal of the same length; 50 083 cu. yd. of material were removed from this slide by dredging from March 1 to March 22. The total net yardage removed under contract by this dredge was 1 211 000, of which 352 000 was for improvement and 859 000 for maintenance.

A 30-in. hydraulic dredge arrived at the Delaware River entrance of the canal on December 22, 1926, and began dredging on maintenance work at the 12.0-ft. contour in Delaware River. It worked west for a distance of 15 200 ft. and began removing the dam constructed by the first dredge at the same time the first dredge was removing the one at Chesapeake City.

The removal of this dam required about three days and the material was deposited in the Delaware City branch of the canal, thus forming another dam to prevent the tide-water from entering by way of that channel. The dredge was moved to St. Georges on February 10, 1927, for improvement dredging at that point, but was delayed to some extent because of the difficulty encountered in removing the steel piles from the coffer-dam. The removal of these and the great amount of timber and débris in the old roadbed required eleven days. The dredge was then moved to the Pennsylvania Railroad Bridge (February 23) and removed the last obstruction to navigation in the canal, thus making it possible to pass through at sea level for the first time.

In addition to dredging done under contract at the railroad bridge, 12 000 cu. yd. of over-burden were washed from back of the piers into the canal to relieve the back pressure; this was later re-dredged. The total net yardage removed under contract by this dredge was 509 000, of which, 87 300 was for improvement and 421 700 for maintenance.

The entire contract was completed on March 22, 1927, after a net total of 1 720 000 cu. yd. of material had been removed and re-deposited in the same spoil areas that had been used under previous contracts. The cost was $568 600, of which $167 500 was for improvement and $401 100 for maintenance.

Cleaning Up.—There were two places in the canal that could not be dredged by the hydraulic method, a bed of boulders 150 ft. long and 50 ft. wide on the south side of the channel at Goose Point, Del., and a part of the old lock at Chesapeake City. For this work, a dipper dredge was leased for a period of 60 days at $500 per day. No great difficulty was experienced in removing the boulders, although some weighed as much as 7 tons. These were loaded on the decks of scows and placed on shore by derricks. In addition to the removal of the boulders and lock wall, several small slides were dredged from the channel, loaded in scows and dumped in front of a 22-in. hydraulic dredge to be rehandled. The total for this item was 11 600 cu. yd. of material (largely rock) removed at a total cost of $32 300.

On March 22, 1927, a slide occurred on the south side of the canal at Guthries Run and a 20-in. dredge was leased for 90 days at $1 500 per day for the purpose of removing this and any other slides that might develop. While the removal of this slide was in progress, the one previously removed and another just west of Summit Bridge were gradually moving channelward and had to be re-dredged.

The canal was completed to project dimensions on May 31, 1927, although it had been provisionally opened to commercial traffic on February 25, 1927. During this time its use had been restricted to daylight periods of high water and to boats of 9-ft. draft and vertical clearance of 45 ft. The canal was formally opened on May 14, 1927. The total quantities and costs involved on all kinds of dredging work is given in Table 1.

LAND SLIDES

Old records indicate that when the canal was originally cut through the summit divide, and for several years thereafter, much trouble was experienced from sliding banks in that particular section, especially in the first mile west of Summit Bridge. (See Fig. 1.) The slides finally became quiescent and remained so until they were again set in motion by the operations of re-construction. Fig. 2 is a view of the original canal. Sliding was undoubtedly caused partly by the practice of the original builders of disposing the spoil very close to the edge of the cut. In some places this spoil was piled up so as to raise the height of the bank from 100 to 110 ft. above sea level; that is, 20 to 30 ft. above the original level. This additional burden undoubtedly contributed to sliding, but the primary cause was the slow deformation of the clay subsoil.

Borings made along the entire length of the canal showed that beneath a top layer of green sand impregnated with some mica, there were various combinations of sand and mud, all of which indicated the decomposition of a heterogeneous collection of rock. Pockets of vari-colored clays were apparently interspersed with similar sands. The results of some borings were interpreted to indicate the presence of quicksand. A Board of Engineers expressed the view[4] (based on the results of borings) that some slips due to water at the upper surface of some of the clay strata, as well as some due to water-saturated material, might be expected during and after the proposed reconstruction. It was fully expected, therefore, that there would be some trouble from sliding banks during and after reconstruction and that they might, at times, interfere somewhat with navigation. The Board believed, however, that slides could readily be handled by following a policy later adopted on the Panama Canal; namely, to provide means of removing them as they developed until the banks became quiescent. A few minor slides did develop during the dredging operations, but owing to the precaution taken to maintain the canal at the high level during the construction period, no damage was done, and traffic was not obstructed by them.

Serious trouble did not develop until after the canal had been lowered to sea level, and again opened to traffic. About ten days after the surface level had been lowered, a section of the south bank near Mile 9 suddenly gave way and partly closed the channel. This was promptly removed by the dredge which had just completed improvement work. In the meantime, several slides were observed to be in motion within the region of the old slides in the first mile west of Summit Bridge, and close enough to the new bridge to cause some anxiety. These moved very slowly, and gave time for a careful examination to be made of their progress and effects. Usually, they were manifest first by the appearance of cracks on the surface of the ground at a distance of 15 to 20 ft. back of the edge of the bank, and parallel with it. The material between the crack and the edge of the bank would then sink almost vertically, the rear portion somewhat more rapidly than the front, so that the original ground surface of this slice of moving material would slope away from the canal. At the same time that this settlement was taking place, there was a general upheaval and breaking up of the material at and near the toe of the slope, extending 15 to 20 ft. above the water surface and encroaching slowly on the channel. The general effect is shown in Fig. 3. After one slice of the banks had settled 8 to 10 ft., a similar crack would appear back of the old one and a new settlement would begin. A series of three or four slices would present the form of a giant stairway.

In addition to the encroachment on the channel laterally, there was also an upheaval of the bottom of the canal, so that both boat traffic and tidal flow would soon be seriously hampered unless dredges were promptly put in operation to remove the invading material. Boring samples were found to be unreliable when checked against the material actually encountered. Excavation showed the material comprising the canal prism in the deep cut to be of three general classes. From the surface down to about Elevation +50 the material was largely a yellowish, loamy sand containing much ferruginous

---

[4] H. R. Doc. No. 391, 62d Cong., 2d Session, pp. 91-93.

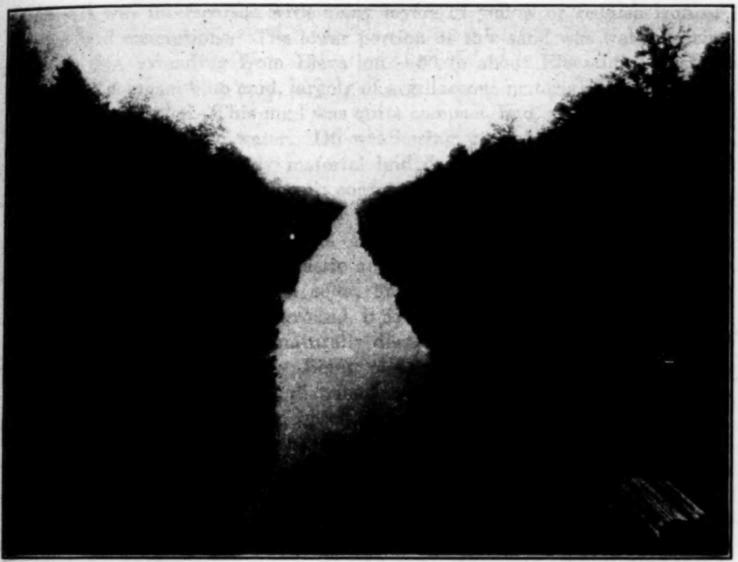


FIG. 2.—PART OF ORIGINAL CANAL IN SECTION C, WHERE SLIDES OCCURRED.

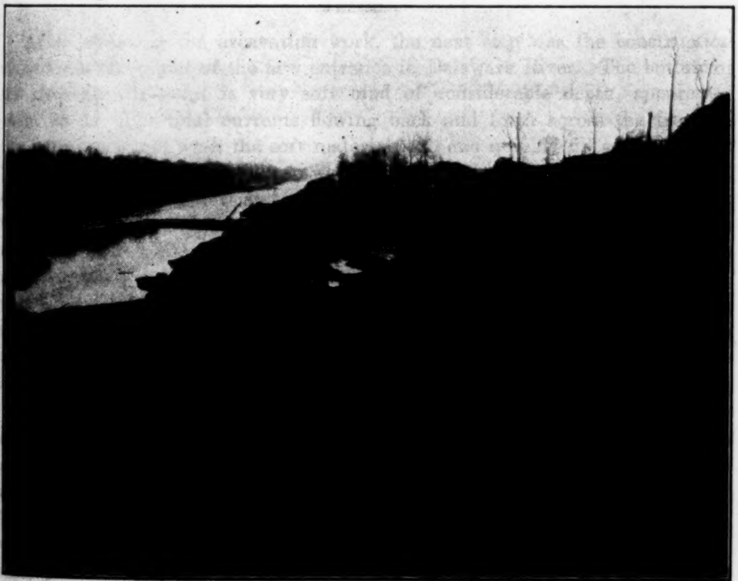


FIG. 3.—VIEW OF SLIDING BANKS IN SECTION C.

matter. It was interspersed with many layers of yellow or reddish iron ore, forming hard concretions. The lower portion of this sand was water-bearing.

Below this, extending from Elevation +50 to about Elevation +5, there was found a compact blue mud, largely of argillaceous material, but containing some siliceous matter. This mud was quite compact, and, although damp, was practically impervious to water. On weathering, it disintegrated into a fine, grayish dust. It is evidently material laid down in a previous era of submergence under fresh water, since it contains no marine fossils.

Immediately below this mud, and showing at about high-tide level, there is a thin layer of lignite which rests on a thick stratum of clay. This underlying bed of clay is moist and plastic and is composed of almost pure kaolin, usually of a whitish or grayish color, but occasionally colored red. Lying 80 ft. below the surface of the ground, it is under considerable pressure. The excavation of the canal prism naturally disturbed the distribution of pressure in the vicinity of the cutting. Being plastic, this material readily changes shape to conform to the new distribution of pressure, and it is this slow deformation of the clay stratum which gives rise to the slides.

No method of preventing the slides is known; drainage would have no effect. It is quite probable that the banks will eventually reach a state of equilibrium as they did formerly; in the meantime, however, much expense will be involved in their removal, and navigation will experience the hazard of occasional interruption. It is fortunate that the section most likely to slide is comparatively short and does not involve the entire deep cut.

JETTIES

After initiating the excavation work, the next step was the construction of jetties at the outlet of the new entrance in Delaware River. The bottom of the river at this point is very soft mud of considerable depth, apparently about 35 ft. The tidal currents flowing back and forth across the front of this entrance would wash the soft material into and quickly fill a dredged cut if it were not prolonged into deep water by confinement between jetties, which concentrate the canal flow and protect the canal cut from the river currents. Studies were made to determine the character of the bottom, and of various types of jetty suitable for the very soft bottom encountered. The decision was to use a rubble-mound stone jetty founded upon a brush mattress. Since brush mattress construction is not well known among contractors in this vicinity, and since the structure was expected to settle an indeterminate distance into the soft mud, it was decided to construct these jetties by hired labor, the brush and stone being purchased by contract.

The two jetties are each approximately 1 350 ft. long and extend from the shore line into a depth of about 25 ft. of water. They are spaced 500 ft. apart at the shore end and 800 ft. apart at the outer end. The tops of the jetties are 10 ft. wide and are at a height of 10 ft. above mean low water.

The brush mattresses extending the entire length of the jetties are 2 ft. thick and 60 to 80 ft. wide. They were sunk as built by a 2-ft. layer of small stone placed upon them, and upon this small stone was placed the larger stone forming the body of the jetty. The large stone varied in size from 10-ton blocks to "one-man" sizes. These jetties, which were completed April 1, 1926, have not yet (November, 1929) shown any signs of settlement. This is an indication of the great efficiency of the brush mattress as a support. The cost of the jetties was approximately $350 000.

St. Georges Creek Tide Dike

The eastern part of the canal between the Pennsylvania Railroad Bridge and a point about 1 mile west of Delaware City occupies the bed of old St. Georges Creek. The new entrance to the canal, starting at Reedy Point on the Delaware River, follows a straight line running westerly in direction until it intersects the bed of the old creek near the former line of the canal at a point about 1½ miles south of Delaware City. In its natural condition, St. Georges Creek extended through a wide marsh area, much of which has been incorporated in the right of way for the canal.

This marsh area was protected from inundation by tides from the Delaware River by levees along the river banks. The opening of the new entrance channel of the canal would have had the effect of admitting the tide into this protected area; hence, in connection with studies for the details of the project, the necessity for modifying it so as to provide protection for the marshes became apparent at an early date. Furthermore, a county road running approximately parallel with the old canal and crossing the marshes (the Dutch Neck Road), was to be crossed by the new entrance, yet this road was not deemed sufficiently important to justify providing a separate bridge. It was decided to provide for these necessities by the construction of a light earthen dike parallel to the new canal on its south side, which would shut the tides out from the remainder of the marshland on the south and would carry the Dutch Neck Road to the Delaware City-Port Penn Road, enabling one bridge to serve both. The construction of this dike had not been provided for in the project document, and, therefore, its cost had not been included in the estimates.

A study was made of the most suitable methods of constructing the proposed dike, and two were considered practicable and economical. It was expected that the dredging contractor might arrange with marshland owners to use some of it for spoil, in which case the spoil itself could be used to construct the bank, but such arrangements could not be consummated. It was then decided to attempt to get a bucket dredge into the marshes for the purpose of constructing the dike by throwing up a suitable embankment, but as the marsh had no outlet by which a dredge could enter, it appeared necessary to construct the new canal entrance up to a point where a supply of tide water could be introduced, and this maneuver was successfully accomplished, with but little effect on the marshlands. The dredge cut its way in through a small narrow channel, which it filled behind itself. The construction of the dike was then conducted under an emergency contract made under proper authority.

This contract was at $3.90 per lin. ft. of completed dike. Work was begun on February 8, 1924, by a bucket dredge which had been pulled over the canal bank and had dug its way down St. Georges Creek to the site of the work.

Progress was slow, however, because the material was mostly vegetable matter and compacted badly. A second bucket dredge was put on the work on May 10, and, by the end of June, the dike was deemed of sufficient height to protect the meadow under ordinary tidal conditions. Operations were then resumed on the new canal on July 7, allowing the tide water from the Delaware River to flow into the low land between the dike and the old canal. On July 16 an extremely high tide topped the dike and caused two breaches which flooded the low land to the south. The bucket dredge continued working on the weak places, but flooding conditions continued to grow worse until September 27, when the Federal Government took over the work of completing the dike. Unfortunately, the material comprising the marshes had proved utterly unfit to support the dike against the pressure of the tides; the weight of the tide water consolidated the bed of the marshes, causing it and the dike to settle so rapidly that this, combined with heavy rains and high tides, soon caused breaches.

Prior to beginning this work it was known that the material comprising the marsh was of a peaty nature of considerable depth, but it was believed that there was a sufficient quantity of mud in it to make it form a substantial dike. Subsequent experience has shown that such was not the case. Moreover, it was exceedingly improbable that any previously projected dike would have been successful, because, lacking previous experience with this material, the design would have been faulty.

The peat comprising the body of this marsh is a spongy vegetable growth, bound together by a matted mass of roots. When a piece of it is separated from its natural submerged bed, it floats. Large chunks occasionally come to the surface and float away. It is also very compressible and acts in a manner similar to rubber. These characteristics made it impossible to build a dike from the dredged material, and better fill had to be brought from the outside. As soon as this heavier material was placed on the marsh it compressed and settled, until, finally, it acted like rubber or a compressed spring. It buckled laterally under the weight and forced the surrounding material up. This action would continue until the entire quantity of the lighter material in a vertical section had been completely replaced by the heavier material.

If these difficulties could have been foreseen before introducing a dredge into the marsh, the heavy material would have been brought in by carts or by narrow-gauge trains. Both these methods were used in subsequent operations, thus supplying data on which to base an opinion of the difficulties that would have attended the construction from the beginning of the entire dike by either method. Carts were fairly successful, but too slow, because enough of them could not dump at the end of the fill to make rapid progress. When parts of the dike turned over, or moved laterally (settling at the same time), no damage was done when carts were used. The use of narrow-gauge trains was not possible until enough solid material had been put in place by other methods to keep the track from sinking, and wrecking the plant.

When the first breaches occurred in the dike, attempts were made to close them by sand-bags. These bags were usually successful in preventing a breach from widening or deepening, but were never successful in effecting a complete closure.

The next attempt at closure was to build the remaining sections of the dike up to a grade safely above high tides by adding sand and other heavy material through bleeders in the pipe lines of the dredges. Then, after all unharmed parts of the dike were thus strengthened and reinforced, the discharge was directed so as to build from the two ends, at the same time pumping as much heavy material as possible into the breach. Two large breaches were finally successfully closed in this manner, but even then the dike continued to settle, and soon became in danger again of being overtopped by high tides. A steam shovel was then procured, with 36-in. gauge locomotives, necessary dump cars, track, etc., and the work of hauling solid material by train from a near-by hill was begun. The cost, including attempts to close breaches, was about $300 000.

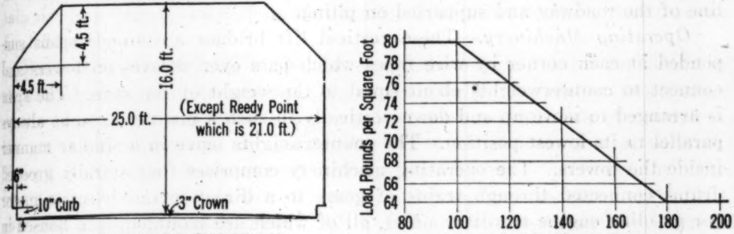
HIGHWAY BRIDGES

The re-construction of the canal required changes in six bridges, one carrying railway traffic and five highway traffic. An additional structure was required over the new entrance south of Delaware City, now called the Reedy Point Bridge, but originally designated the New Cut-Off Bridge. There were wooden jack-knife swing bridges at four of the points, and those at Chesapeake City, Summit Bridge, and Saint Georges were on highways vital to traffic on the peninsula. The Summit Bridge was an iron horizontal swing drawbridge. By arrangement with the State of Maryland it was decided not to replace the structure at Bethel, known as Maryland Pivot, which was removed as soon as dredging operations made it necessary. Bids for a new bridge over the part of the old channel in Delaware City proper were deemed unduly high and it was decided not to construct a new bridge there, since traffic did not require it, but to alter the foundations of the existing structure at that point and to strengthen it sufficiently for present needs.

The new bridges were designed to serve the canal, not only as constructed under the present project (that is, at sea level with a 12-ft. depth and 90-ft. bottom width), but also as projected for the future, dredged to tide-water with a depth of possibly 30 ft. and a bottom width of 150 to 175 ft., with provision for a horizontal clearance of 175 ft. and a vertical clearance of 140 ft.

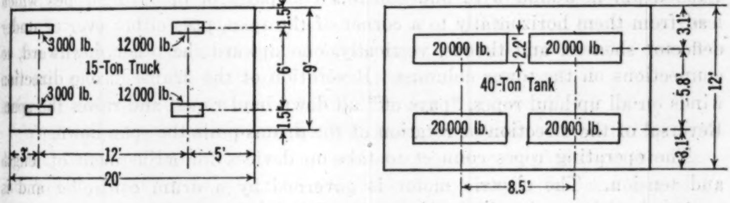
The vertical lift type of bridge was selected for all locations on the basis of lower first cost and maintenance expense. Furthermore, fireproof floors could thus be provided economically, and most of the vessels would require the bridge to be only partly open, thus reducing further the operating expenses and delays to land traffic. Operation is by direct current from storage batteries that are charged by a motor generator set using commercial alternating current; emergency operation is by gasoline engine which can either drive the generator or raise the span by direct connection through a belt to the gear train.

All bridges are of the through truss type and were designed to accommodate highway traffic adequately for future years with the continued growth of motor transportation, and to carry safely loads incidental to troop movements. The clearance diagram and design loads for the bridges at Chesapeake City, Summit, and St. Georges, are shown in Fig. 4. Sidewalks 4 ft. 6 in. wide were provided outside the trusses. The roadway clearance of the Reedy Point Bridge is 4 ft. narrower than that of the other three; otherwise, the clearances are the same.


4.5 ft.
16.0 ft.
4.5 ft.
25.0 ft.
(Except Reedy Point which is 21.0 ft.)
6"
10" Curb
3" Crown
Load, Pounds per Square Foot
80
78
76
74
72
70
68
66
64
80
100
120
140
160
180
200
Length of Span in Feet

(a) CLEARANCE DIAGRAM

(b) UNIFORM LOAD FOR TRUSSES


3000 lb.
12000 lb.
15-Ton Truck
3000 lb.
12000 lb.
1.5'
6'
9'
1.5'
3'
12'
5'
20'
20000 lb.
2.2'
20000 lb.
3.1'
40-Ton Tank
5.8'
12'
20000 lb.
20000 lb.
3.1'
8.5'

(c) CONCENTRATED LOAD FOR FLOORS

FIG. 4.—LOADING DIAGRAMS AND CLEARANCES FOR BRIDGES AT CHESAPEAKE CITY, SUMMIT, AND ST. GEORGES, CHESAPEAKE AND DELAWARE CANAL.

Loading.—The live loads used for design were as follows:

For the floor system of the Chesapeake City, St. Georges, and Summit Bridges, two 15-ton trucks abreast, or uniform load, as shown in Fig. 4(*a*), or one 40-ton tank.

For the floor system of the Reedy Point Bridge, two 15-ton trucks abreast, or uniform load as shown in Fig. 4(*b*).

For all sidewalks, 100 lb. per sq. ft.

For all trusses, uniform live load, as shown in Fig. 4, over the entire roadway plus one-half this uniform load on the sidewalks, or the loadings specified for the floor system.

An impact allowance of 30% was made for all live loads.

Canal traffic was required to be maintained uninterrupted during the construction of any bridge; hence, the specifications called for a vertical clearance of 55 ft. for a width of 25 ft. to be maintained in all falsework over the channel.

Since the ultimate canal will have a depth below low water of possibly 35 ft. and since no rock was found in any exploration borings, the least depth to which pier bases might safely be carried was considered to be Elevation —40. The plans in each case stipulated that caissons be sunk either by open dredging or by the pneumatic process at the option of the contractor, and that in either case they must be unwatered for examination of the foundation material before sealing. The contractor elected to use a caisson of his own design on all contracts and to sink it by open dredging until such time as this method became impracticable, and then to resort to the pneumatic method.

Except at Summit, where the so-called abutments are really buried piers, the abutments are of conventional type with wing-walls parallel to the center line of the roadway and supported on piling.

Operating Machinery.—These vertical lift bridges are simple spans suspended at each corner by wire ropes which pass over sheaves on towers and connect to counterweights about equal to the weight of the span. The span is arranged to move up and down vertically in such a manner as to be always parallel to its lowest position. The counterweights move in a similar manner inside the towers. The operating machinery comprises four spirally grooved drums connected through trains of gears to a direct-current electric motor (or gasoline engine auxiliary unit), all of which are mounted on a housed-in platform at the center of the lift span and just above the roadway clearance. Each drum is wound with and controls two pairs of operating ropes which lead from them horizontally to a corner of the span, pass either over or under deflector sheaves, and thence, vertically, one upward the other downward, to connections on the tower columns. Revolution of the drums in one direction winds on all up-haul ropes, "pays off" all down-haul ropes, and raises the span. Reversal of the direction of rotation of the drums pulls the span down.

The operating ropes connect to take-up devices for adjustment of length and tension. The electric motor is governed by a drum controller and is equipped with a solenoid brake, with a hand-brake as auxiliary equipment. Thus, the span is moved, held level during operation, and may be stopped and held at any point by stopping the machinery.

When a span is raised, say, 90 ft., there is transferred from the span side of the tower sheaves to the counterweight side, 90 lin. ft. of suspending ropes at each corner of the span. This is no inconsequential weight and would materially affect the balance of the span were it not compensated. For the short lift at Summit (68 ft.) this change in balance is not dangerous; nor does it cause excessive power consumption. Compensation is obtained on the other three bridges by balance chains hung between each counterweight and its tower so that as the span rises and is lightened by the loss of suspending ropes on its side of the sheave, double this weight of the chains is removed from the counterweights. The balance thus remains unchanged for any position of the span.

Use of Concrete.—Each counterweight is a block of concrete with pockets or wells at the top for balance blocks. A structural steel girder is used to carry the weight of the forms and the first, or girder, layer of the counterweight itself. This girder is a heavily reinforced concrete beam designed to carry the superimposed concrete of the completed counterweight and, therefore, it was allowed to cure thoroughly before any further concreting was done on the counterweight. The forms used were of $\frac{5}{16}$-in. and $\frac{9}{16}$-in. steel plates held to proper width and length by $\frac{5}{8}$-in. tie-rods. Difficulty was experienced in obtaining plane faces because no wales or continuous stiffeners were used and the sheets were too thin to have the necessary stiffness.

Materials and Concreting.—All materials were delivered by barge. Sand and gravel were stored on plank platforms. Water for the concrete was taken from the upper level of the canal for all jobs except that at Reedy Point, where wells were sunk to avoid the use of brackish tide water. The specifications called for the concrete to be proportioned by volume of materials, but to contain not less then 6 sacks of cement for 1 yd. of 1:2:4 concrete, nor 5 sacks for 1:2.5:5 concrete.

Some variation occurred in the gravel and there were natural variations in the sand due to moisture content. It was the practice to vary the proportions of the aggregates for workability and to keep within a 0.1 sack of cement of the prescribed minimum, always keeping the water ratio as low as possible. Except at Summit, measurement was by wheel-barrow. However, by calibrating the bucket into which the mixer discharged, a constant check was available on the cement content. After each day's run the sack count was checked against the concrete yardage placed in the forms. At Summit Bridge, conditions permitted two whirlers to operate within reach of one mixer, and a batcher was built which permitted the use of a method that proved superior to the antiquated one used on the other jobs; one whirler handled concrete and the other fed the batcher. Water regulation and control were rigidly enforced, and the concrete never showed a slump test exceeding 2 in.

When the temperature was below 40° Fahr., the water was heated by turning the mixer exhaust into the tank and steam jets were placed in the aggregate piles. Fresh mass concrete was covered by tarpaulins and heated by steam. All materials were unloaded to stock by whirlers which also handled the concrete between the mixer and the forms.

Tower Straightening.—In bridges of this type the entire load of the span and counterweights is carried by the front legs of the towers, the rear legs acting only as bracing members to carry dead and wind loads. When the load is applied to the front legs or columns of the towers, there is an appreciable shortening due to the elasticity of the metal; consequently, these legs are fabricated longer than normal in order to compensate for this shortening under load. As this shortening takes place there is a progressive forward rotation of the tower about each rear-leg panel-point, which extends to the bottom. In order to have these tower columns plumb in the completed bridge they are erected in a cambered position. The towers must be plumb in the transverse direction so that the span will hang free of its guides and thus operate with a minimum power demand. Furthermore, both columns of the tower at the fixed end of the lift span must be in the same vertical plane to prevent binding of the jaw-guides which hold the span against longitudinal movement. The heights of the towers from the top of the masonry to the center line of the sheaves and the extreme span lifts are as given in Table 2.

The towers of the St. Georges and Chesapeake City Bridges were plumbed by using plumb-bobs and those of the Summit and Reedy Point Bridges, by transit. Each method has its advantages and either will give accurate results under proper working conditions.

Electrical Equipment.—The electrical units installed on the various jobs are as listed in Table 3.

Illumination.—All illumination is by electric power from the storage battery. There are lights in the battery house and in the operating room, on the stairs to the operating room, on the towers at the foot and top, for roadway illumination, as required by law for navigation, warning signals on the barrier gates, and flashing signals at the portals. As these lights are on wiring circuits regularly carrying the normal battery voltage and as the voltage is much higher during the time when the battery is being charged, variable resistances are installed in all lighting circuits to permit reducing the voltage to a safe maximum.

TABLE 2.

Bridge.	Height of tower, in feet.	Height of lift, in feet.
St. Georges, Del.	168.1	123
Chesapeake City, Md	186.0	131
Summit Bridge, Del.	115.6	71
Reedy Point, Del	174.1	131

Telephone.—All bridge operating rooms are connected by telephone to commercial exchanges regardless of the position of the span. This is accomplished by means of a flexible cable between the span and one tower. The suspension point on the tower is slightly above the mid-height. A small sheave riding in the loop of the cable and held by a vertical guide-wire restrains the cable from blowing against the side of the tower, or under the span.

TABLE 3.—LIST OF ELECTRICAL UNITS AT THE VARIOUS BRIDGES.

Units.	St. Georges, Del.	Chesapeake City, Md.	Summit, Del.	Reedy Point, Del.
Total moving load, in tons.	1 425	2 040	1 400	1 138
Operating motor (horsepower)	70	100	70	70
Rates of speed in rising, in feet per minute	65	75	30	70
Time for full opening, in minutes	2	2	2	2
Number of cells in battery	250	250	110	250
Type of cells	F 9	F 13	F 13	F 9
Voltage of battery	520	520	230	520
Capacity of battery	40 amperes for 8 hours; final voltage, 437; 160 amperes for 1 hour; final voltage, 405.	60 amperes for 8 hours; final voltage, 437; 240 amperes for 1 hour; final voltage, 405.	60 amperes for 8 hours; final voltage, 192; 240 amperes for 1 hour; final voltage, 178.	40 amperes for 8 hours; final voltage, 437; 160 amperes for 1 hour; final voltage, 405.

Warning Signals to Highway Traffic.—For signaling to highway traffic that the bridge is being raised there is a warning bell and a pair of flashing red lights on the oncoming side of each approach. These are put in action before the opening is begun, by turning a snap switch in the operating house. The switch must be on before power is available for operating the motor.

Storage Batteries.—The storage batteries are housed in well-ventilated brick buildings near the north end of each bridge. On all bridges except that at Reedy Point an ampere-hour meter in the battery leads registers the approximate quantity of stored power at all times. The indicator hand moves in one direction as power is withdrawn and returns as it is replaced by charging. An automatic trip on this meter shuts the generator set down as soon as the battery reaches the fully charged state.

Details of Construction, St. Georges Bridge.—The plans for this bridge required a symmetrical layout of the two piers and two abutments, as shown in Fig. 5. The contractor worked the north side of the canal with a 10-ton "whirler" derrick equipped with a 75-ft. boom and the south side with a steel stiff-leg derrick. The caissons were 15 ft. by 41 ft. in plan. The north one was to be driven to Elevation — 43.0 and the south one to Elevation — 41.0; in each case the top of the base was at Elevation 0.0, at which point the neat work begins. An attached coffer-dam was required because this bridge crossed the canal in the upper level where the water was held at Elevation + 17.5. The cutting-edge of the north caisson was set at Elevation + 9.0. The first concrete poured was allowed to set six days before the forms were stripped and the sinking was begun.

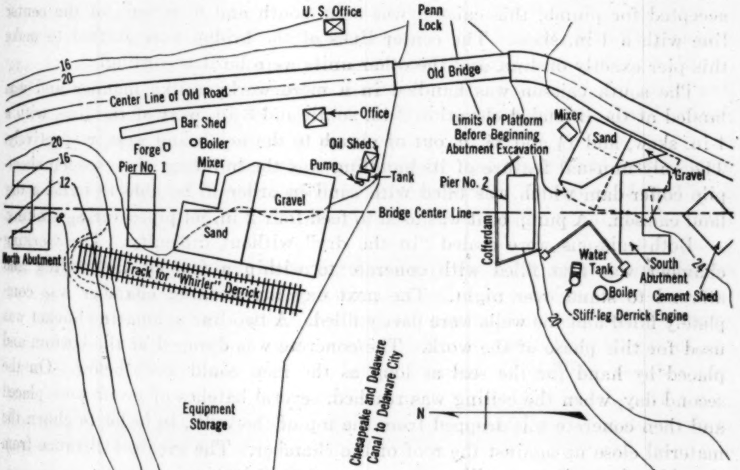

U. S. Office
Penn Lock
16
20
Old Bridge
Center Line of Old Road
Office
Bar Shed
Boiler
Limits of Platform Before Beginning Abutment Excavation
Mixer
Sand
20
16
12
Forge
Pier No. 1
Mixer
Oil Shed
Pump
Tank
Gravel
Pier No. 2
Gravel
Gravel
Bridge Center Line
Sand
Cofferdam
Water Tank
South Abutment
24
North Abutment
Track for "Whirler" Derrick
Boiler
Cement Shed
Stiff-leg Derrick Engine
Chesapeake and Delaware Canal to Delaware City
Equipment Storage
N
Cement Shed

FIG. 5.—PLANT LAYOUT OF SUBSTRUCTURE, ST. GEORGES BRIDGE.

Sinking was halted when the top of the concrete was about 2½ ft. above the ground surface, forms were set, and another lift of concrete was poured. These lifts averaged about 6 ft. in height. Forms were removed in not less than 48 hours and sinking was resumed. No unusual sinking difficulties were encountered in either of these caissons except that in the beginning the contractor's organization was indifferent to the need of checking the location adequately and directing the mucking properly. The inspection force was determined to make the pioneer job a standard for the following ones, however, and the contractor was required to plumb the north caisson which was leaning 12½ in. north and 11½ in. west, with only 4 ft. to sink before the landing elevation was reached. The cutting-edge at this point was 48 ft. below the ground surface and the material had been quite firm for the 27 ft. of sinking. The center of gravity was well below the ground line and mucking under the high edge combined with blocking under the low edge was ineffectual to right the caisson, even when two water jets were worked along the high side. All this time the caisson was sinking and was soon below the intended landing point and in acceptable material for sealing. Finally, a bridle was rigged on needle-beams and twice daily a strain was taken and clamped off, and the jetting was continued. By this method in 13 days the caisson was brought to within 2 in. of plumb in this direction. In the transverse direction the high end was loaded with earth by covering the east well and sheathing the east row of struts, thus forming a pocket inside the coffer-dam. Two jets were worked around the high end and the leaning was corrected from 12 to 1½ in. in 11 days. When accepted for plumb, this caisson was 3 in. south and 5 in. east of the center line with a 1-in. skew. The center lines of the bridge were shifted to make this pier exactly on line, and the other units were built accordingly.

The south caisson was handled in a more workmanlike manner and was landed at the intended elevation 2 in. south and 3 in. west of the line, with a 1-in. skew, and 1½ and 2½ in. out of plumb to the north and west, respectively. The only unusual feature of its handling was the building of a wooden sheet-pile coffer-dam which was filled with sand in order to be able to build a dry land caisson. A pump boat was used to feed four 2-in. jet pipes during sinking.

Both caissons were sealed "in the dry" without difficulty. The working chamber was first filled with concrete to within a foot of the ceiling and allowed to stand over night. The next day the working chamber was completely filled and the wells were partly filled. A two-line submarine bucket was used for this phase of the work. The concrete was dumped at the bottom and placed by hand for the seal as long as the men could work below. On the second day, when the ceiling was reached, several batches of grout were placed and then concrete was dropped from the top of the wells, in order to churn the material close up against the roof of the chamber. The greatest distance from the side of a well to a ceiling corner was 2 ft. 3 in.

Piers.—The neat work on the piers was of conventional dumb-bell outline and presented no unusual problems. The shafts, recessed for the web walls, were poured first, and after the concrete was rubbed, the coffer-dam was braced against the shafts to permit the removal of the cross-braces so that the web wall might be poured without boxing out the bracing timbers. After this work was finished, the bolts holding the sheeting of the attached coffer-dam to the caisson were backed out, the timber bracing was removed just ahead of the back-filling, and the sheeting pulled.

Abutments.—The abutments are supported on concrete piles 14 in. square and varying in length from 16 to 26 ft. The lengths necessary were determined by driving a timber test pile of the same cross-section (14 in. by 14 in.) as the concrete piles, and 30 ft. long; these test piles were driven when the abutment excavation was at such a depth that the little additional material penetrated might be disregarded. Seven of these piles were driven for the north abutment and a contour sketch was made of the elevations of the tips when the requisite bearing of 20 tons per pile was reached. On this basis lengths were determined for pre-casting the piles. The first piles driven stopped at the depths anticipated, but those driven later, after the soil became more compacted, met with greater resistance. It was impossible to drive test piles even to the minimum penetration required in the coffer-dam at the south abutment. A 1½-in. jet was rigged up and five holes jetted over the area disclosed the presence of a thick layer of fine tight blue sand through which the piles would have to be jetted. Consequently, 20 ft. 3-in. piles were cast with a 2-in. pipe in the center. In driving, a hole was first made with a free jet and the pile placed therein with its inside jet flowing. At times, the outside jet was required during driving. In all cases the water was shut off when the pile was within a foot of the required depth and the remaining penetration was secured by the hammer alone, in order to have a figure for computing its supporting capacity.

The coffer-dams for these abutments were of conventional design. Tongue-and-groove sheeting was used on one and a double thickness of 2-in. by 10-in. and 2-in. by 12-in. oak sheeting for the other. The bottom of the main wall footing was at Elevation + 0.5, or 17 ft. below the water level in the old canal, and, therefore, constant pumping was required.

The piles were cast during the winter months and were cured under steam at about 120° Fahr. They withstood driving very well. Two piles of 1 : 1.5 : 3 concrete, cured for 19 days under steam, were driven on the twentieth day with only a small amount of spalling on the head of one. A No. 2 Vulcan hammer was used for all the driving; swinging leads with three guys were used and the bottom of the pile was accurately set for locating it in position.

The neat work of the abutments was of conventional design. By some overhead bracing and tying back it was possible to remove the struts of the coffer-dam timbering and pour the walls without boxing out the bracing. Material for the approach embankment was partly caisson spoil, partly clay dug by an orange-peel bucket from the canal bottom, and partly material from borrow-pits at the bridge site, but mostly it was material borrowed from pits near-by.

Erection of Steel Work.—The two towers and the lift span of St. Georges Bridge contain about 250 tons of metal each. With the floor, the lift span weighs 700 tons. Due to the necessity of not interfering with canal traffic and the fact that five panels of the lift span were over dry land prior to the final dredging operations, it was necessary to erect the span on falsework high enough to provide 55 ft. of vertical clearance over the water. All steel for this contract was transported by water and unloaded at the site by a stiff-leg derrick on the north side, which was mounted to travel on rails parallel to the bridge (see Fig. 6). This derrick had an 85-ft. boom. Steel for the south tower was then carried across the canal on a scow to within reach of the derrick.

The bottom section of the north tower was erected by means of the stiff-leg derrick which then was used to erect the falsework for the north five panels of the lift span. A steel guy derrick with a 75-ft. mast and a 70-ft. boom, supported on falsework 117 ft. high, was then rigged to complete the erection of the north tower (see Fig. 6) and, after being moved to a lower position, was used to erect the north five panels of the lift span. This derrick was then moved to the south side of the canal for setting the steel in the bottom section of the south tower. Its next position was on falsework inside the tower, where the bottom of the mast was about 36 ft. above the top of the masonry. From this point a falsework bent was built to carry the south end of the lift span. During this time a "jinniwink" working from the top chord of the lift span, and fed by the stiff-leg derrick, placed the steel of the two panels south of the center panel, cantilevering over the channel. The guy derrick then completed the lift span and south tower in two moves.

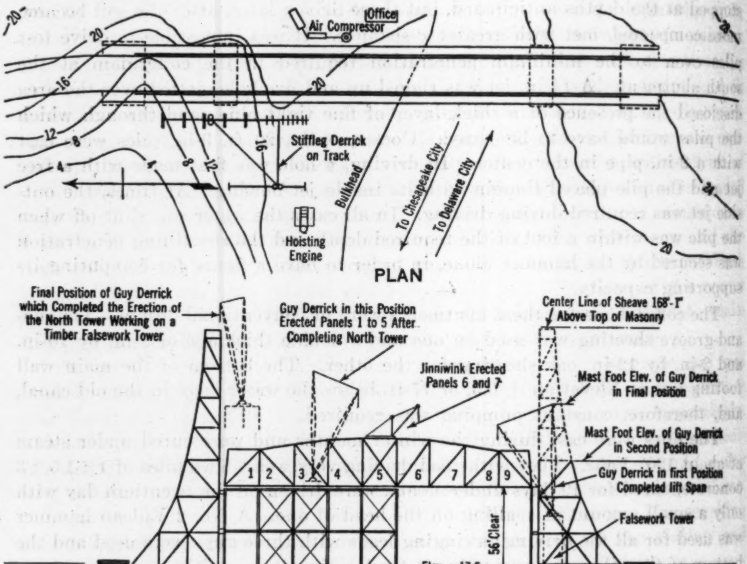

20
20
16
12
8
4
Office
Air Compressor
20
16
12
8
4
Stiffleg Derrick on Track
Hoisting Engine
Bulkhead
To Chesapeake City
To Delaware City
20
20
20
PLAN
Final Position of Guy Derrick which Completed the Erection of the North Tower Working on a Timber Falsework Tower
Guy Derrick in this Position Erected Panels 1 to 5 After Completing North Tower
Jinniwink Erected Panels 6 and 7
Center Line of Sheave 168'-1" Above Top of Masonry
Mast Foot Elev. of Guy Derrick in Final Position
Mast Foot Elev. of Guy Derrick in Second Position
Guy Derrick in first Position Completed lift Span
Falsework Tower
1
2
3
4
5
6
7
8
9
56' Clear
Elev. + 17.5
Canal
Center Line Pier Base
Center Line Pier Base
Face of Backwall
61'-3"
210'-0"
61'-3"
Face of Backwall
WEST ELEVATION

FIG. 6.—JOB LAYOUT, ST. GEORGES BRIDGE SUPERSTRUCTURE.

Steel-plate counterweight forms were assembled on the floor of the approach spans with the reinforcing and counterweight girders inside. Two counterweight ropes were passed over each sheave, by a gin-pole mounted on top the tower, and connected through the equalizers and eye-bars to the counterweight girder. The opposite ends of these ropes were then connected to two sets of blocks reeved for two lines to a hoisting engine, and the counterweight box with the girder was raised until the ropes could be attached to the lifting girder. The gin-pole then placed the remaining ropes over the sheaves. After all the counterweight ropes had been connected and allowed to carry the full load of the empty counterweight box (about 20 tons) some variation in length was observed, due to unequal untwisting of the ropes in erecting. These long ropes were disconnected at one end and were reconnected with from one-half to one and one-half turns more twist in them, depending on the shortening desired. Tests made later at the rope factory on 1¾-in. ropes, 176 ft. long, showed that a half turn changed the length nearly ½ in. The balance chains were erected in short sections after the counterweight ropes, using a line from the hoisting engine.

The towers were plumbed in the position of computed camber before riveting was begun. The lift span was swung clear of the falsework before it was riveted.

Concreting.—Concreting of the floor-slabs and counterweights was done in the following sequence, in order not to overload the two falsework bents which were then the sole support of the lift span: (1) North span slabs; (2) 7-ft. pour at the north counterweight; (3) 7-ft. pour at the south counterweight; (4) lift-span slabs; (5) counterweights completed; and (6) south span slabs.

Due to cold weather, a special quick-setting cement was used for the sidewalk slabs of the south span. All concrete, of course, was heated prior to placing in the forms. The metal counterweight forms were draped with tarpaulins extending well below the bottoms and were heated by steam jets. Concrete for the counterweights and the lift span slabs was elevated by means of a bucket running up a mast. Such a mast was set up on the floor of each tower span.

Adjustments.—The bridge is paved with 5-in. by 12-in. asphalt blocks laid on a ½-in. mortar cushion. By the time the paving was completed, all machinery, other than electrical, was installed, and the counterweights were loaded with balance blocks until the span could be raised by the gasoline engine. The falsework bents were then removed and the span was lowered to the down position; the shoes were grouted to proper elevation; the expansion plates on the fixed spans were set to match those on the lift span; and the span and the counterweight guide casting were set and attached. One of the castings had been attached prior to the pouring of any counterweight concrete. After beginning operation, it developed that that method was improper because the counterweight did not hang exactly as had been expected and the guide casting wore quite rapidly. It had to be replaced by one properly set.

Traffic was turned on the bridge on March 1, 1926, to permit the removal of an old lock, and the bridge was operated by the auxiliary power unit pending the completion of electrical installation. Since the movement was only 10 ft.

of travel per min., much adverse comment was forthcoming from the traveling public.

The installation of electrical equipment was completed before commercial power was made available for operating the motor generator set for charging the battery. After the battery (250 cells of 9 plates each) was set up and the electrolyte poured, an attempt was made to make the initial charge by driving the generator by the gasoline motor, but to charge the entire battery as a unit was impossible, because the belt drive, from engine to generator, was too short and was in the wrong direction to transmit full load. Consequently, the battery was split in half for charging purposes and each half was charged for twelve hours each day continuously except for time required for making lifts. Ten days were required to raise the specific gravity to a point high enough to make it safe to leave the battery inactive. When commercial electric power was made available, operation was begun with the use of regular equipment. The speed of travel in lifting was increased to 60 ft. per min., with a satisfactory shortening of delay to traffic. The cost of this bridge was $354 000.

SUBSTRUCTURE OF THE CHESAPEAKE CITY BRIDGE

At Chesapeake City the bridge crossing was over Back Creek, which is tide water. Both pier locations were in the stream and both abutment locations were covered at high tide. The contractor elected to work each side with his favorite bit of equipment, a 10-ton "whirler" derrick with a 75-ft. boom. (See Fig. 7.)

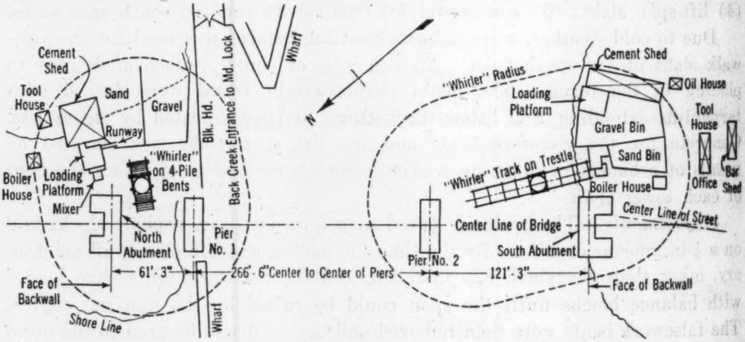

Cement Shed
Tool House
Sand
Gravel
Runway
Blk. Hd.
Back Creek Entrance to Md. Lock
Wharf
"Whirler" on 4-Pile Bents
Boiler House
Loading Platform
Mixer
North Abutment
Pier No. 1
Face of Backwall
Shore Line
61'-3"
266'-6" Center to Center of Piers
Wharf
N
"Whirler" Radius
Cement Shed
Oil House
Loading Platform
Gravel Bin
Tool House
Sand Bin
"Whirler" Track on Trestle
Boiler House
Office
Bar Shed
Center Line of Street
Center Line of Bridge
South Abutment
Pier No. 2
121'-3"
Face of Backwall

FIG. 7.—PLANT AND JOB LAYOUT, CHESAPEAKE CITY BRIDGE SUBSTRUCTURE.

Caissons.—The pier bases were 16 by 43 ft. in plan and were built 42 ft. and 49 ft. high for the north and south piers, respectively. The planned elevation of the tops was —15 (mean low water at Chesapeake City is Elevation +2.3 and mean high water is Elevation +4.5). Floating caissons 26 ft. high were built on land ways and launched at high tide. A composite steel and timber construction was used; the cutting-edge, walls, and roof of the working chamber and wells for a height of 20 ft. were of steel-plate and angle construction as was some bracing in the bottom 6 ft. The outside faces above the cutting-edge were of 3 by 12-in. tongue-and-groove oak sheeting supported by 12 by 12-in. timber bracing.

Both caissons were adequately caulked and no difficulty was encountered in towing them to position. The work of sinking the floating caisson to the creek bottom was begun on an ebbing tide until 6 ft. of concrete had been poured. At first, difficulty was experienced in keeping the caissons plumb because the 1-yd. concrete bucket was too large for the narrow caisson openings. Open-well dredging was then begun and concrete was poured as required. Coffer-dams, similar in detail to those used at St. Georges, were attached above the tops of the caissons.

The south caisson was sunk by open-well dredging until it became apparent that proper control was impossible because of buried logs. The equipment for pneumatic work was then installed. There were three wells, 7 ft. in diameter, and one man-lock, one muck-lock, and one concrete lock. This caisson was sunk under air from Elevation —28 to Elevation —42, when it was necessary to shift the pneumatic equipment to the north caisson. Open dredging was again attempted and abandoned as unsatisfactory. After the north caisson was sealed, the air equipment was returned to the south caisson, which was uneventfully driven to Elevation —57 and sealed in a hard blue clay. For the sealing, both the concrete lock and the muck-lock were used; the working chamber was filled to within 1 ft. of the ceiling and allowed to set under pressure for 60 hours before removing the locks and concreting the wells.

The north caisson was sunk to Elevation —42 by open dredging, where strata of sandstone prevented further sinking by this method and the pumps could not keep the water down sufficiently for men to enter the working chamber. The air plant was installed and the sinking was continued to Elevation —64. The material at this elevation was not as good as was desired and, after making a series of explorations by auger borings, it was decided to stop the caisson where it was, but to excavate to Elevation —69 before sealing.

Erection of Steel.—The site of the Chesapeake City Bridge and the suitable plan of erection differed radically from those of the other jobs. All three spans were over water and within reach of the boom of a derrick-boat. Furthermore, the span was too long to erect, economically, on falsework, and, at the same time, provide the required clearances.

The towers, above the bottom section, were erected by a creeper which traveled up the front legs. A small stiff-leg derrick (equipped with a jib) was mounted on a trussed frame that rested on brackets bolted to the tower columns. The entire unit was hoisted by cables reeved through blocks attached to the tops of the last erected column sections. The sheaves were set and the ropes hung by this creeper.

The counterweight forms, of the same metal arrangement as those at St. Georges, were assembled resting on a grillage supported by temporary brackets attached to the tower front and rear legs. In this position the counterweight was about 1½ ft. below its position with the span seated. After the towers were riveted, the counterweight concrete was poured sufficiently high in the forms so as not quite to balance the metal and machinery of the lift span.

The balance chains could not be attached until after the supporting grillage had been removed.

The lift span was erected on three scows anchored in front of the south pier and was protected by a system of pile clusters. The operating machinery was installed and the down-haul ropes run on the drums. The up-haul ropes had been attached to the tower tops. The falsework on the scows had been detailed so that, at high tide, the span would be high enough to permit attaching the ropes, already fastened to the counterweights, to the lifting girder and, by partly flooding them at low tide, the scows would lower the span to its position on the shoes and be low enough to pull out from under.

On January 23, 1926, canal traffic at this point was suspended and the lift span was floated into a position where it should have been possible to move it sidewise into place over its shoes. An interference developed which required the removal of four deflector sheaves and a steel member in the roadway floor system at the north end before the span was coaxed into position. The counterweight ropes (sixty-four in all) were connected and after the scows were taken from under the span, the supports upon which the counterweights had been built were removed, and the installation of operating ropes was completed. Thereupon, an effort was made to raise the span by means of the 40-h.p. gasoline engine. The first effort was unsuccessful, due to extra weight that had been placed on the span to insure its picking the counterweights off their supports. After the removal of some of this weight, the span was picked on the second attempt with the engine fully loaded. As the span was raised and the weight of the counterweight ropes was transferred from the span to the counterweight side of the sheaves the engine found the load easier and easier to pull until after having lifted the span 86 ft. the balance was nearly even. The span was lashed at this point and was not moved again until all the concrete and paving work was completed.

The approach spans were riveted as soon as they were swung and the towers as soon as they were plumbed and before the counterweights were poured. On the lift span the lifting posts, lifting girders, floor-beam connections, and stringer connections at alternate floor-beams were riveted while the scows were carrying the load; all other riveting was done after the span was swung.

After the span had been raised and canal navigation had been resumed, an investigation as to the cause of the lack of end clearance during the "floating in" was made. It was discovered that through an error in the drafting-room, four members in the eleven-panel trusses, had been fabricated 2⅝ in. too long. To replace them was impracticable and it was necessary to shift all the deflector sheaves 2⅝ in. and to reinforce all top chord splices (many of which were partly open) so that the stresses might be computed as being carried entirely by rivets.

On April 24, 1926, all metal work, concrete, paving, and machinery installation had been completed, and the bridge was opened to traffic to allow the removal of the old bridge. On the next day, while operating in a heavy wind, a balance chain fouled on a horizontal brace of the north tower, and broke. The falling chain and the tower were severely damaged. The bridge was closed to traffic and replacement parts and material for a revision of details of the chains and towers were ordered. The approaches to this bridge were constructed by the State of Maryland. The cost of the bridge was $476 000.

SUBSTRUCTURE OF SUMMIT BRIDGE

Summit Bridge is located 5 miles east of Chesapeake City, in what is known as the "Deep Cut". The banks at this point are 80 ft. high and it was necessary to excavate 56 000 yd. of dry material so that the substructure could be put in and the approaches made. This was an extra contract item and was performed by steam shovel and trucks in a period of four months, before actual work on the regular contract could begin. The bridge rests on four piers as shown in Fig. 8. Piers Nos. 1 and 4 are spill-through abutments, and Piers Nos. 2 and 3 are essentially of the dumb-bell type, resting on solid concrete bases.

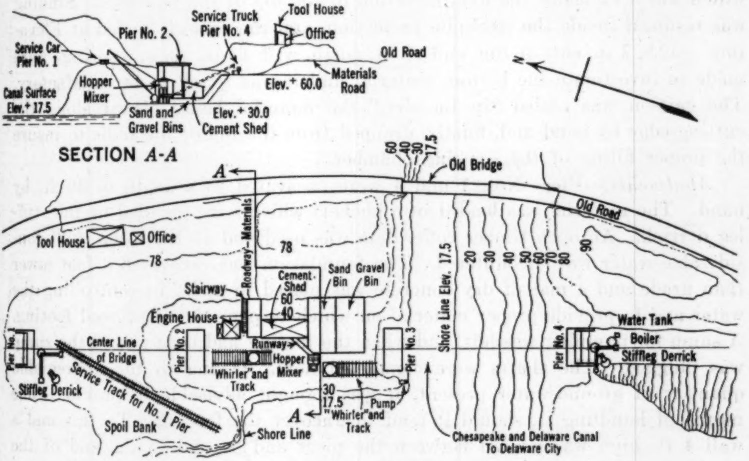

Service Car Pier No. 1
Pier No. 2
Service Truck Pier No. 4
Tool House
Office
Old Road
N
Canal Surface Elev.+ 17.5
Hopper Mixer
Sand and Gravel Bins
Elev.+ 30.0
Cement Shed
Elev.+ 60.0
Materials Road
SECTION A-A
A
60
40
30
17.5
Old Bridge
Old Road
Tool House
Office
78
Roadway-Materials
78
Stairway
Cement Shed
60
40
Sand Bin
Gravel Bin
Shore Line Elev. 17.5
20
30
40
50
60
70
80
90
Engine House
Runway
Hopper Mixer
Water Tank
Boiler
Stiffleg Derrick
Pier No. 1
Center Line of Bridge
Pier No. 2
Pier No. 3
Pier No. 4
"Whirler" and Track
Stiffleg Derrick
Service Track for No. 1 Pier
30
17.5
Pump
"Whirler" and Track
Spoil Bank
Shore Line
Chesapeake and Delaware Canal To Delaware City

FIG. 8.—PLANT AND JOB LAYOUT, SUMMIT BRIDGE SUBSTRUCTURE.

The contractor used two 10-ton "whirlers" with 75-ft. booms for Piers Nos. 2 and 3, and two stiff-leg derricks for Piers Nos. 1 and 4. A central mixing plant was used for concreting. The two "whirlers" handled materials and placed the concrete for Piers Nos. 2 and 3, but trucking was necessary to convey the concrete to Piers Nos. 1 and 4.

Caissons.—The pier bases were built by sinking concrete caissons, 20 by 46 ft. in plan, with three 7-ft. wells. These wells are of the same type as those described for the bridge at St. Georges. The base for the south pier was planned for a height of 49 ft., but was built 55 ft. high. The base for the north pier was planned and built with a height of 43 ft. The attached cofferdams were similar in detail to those at St. Georges Bridge.

The shoe for Pier No. 2 was set at Elevation +41 and the first pour made. After the usual 6-day curing time, the work of sinking the caisson was begun. At Elevation —27 it became necessary to extend the attached coffer-dam (which was already 24 ft. high) by an additional 20-ft. section. The sinking of the caisson was resumed, but friction on the sides of the attached coffer-dams became so great that three days of jetting and jarring with dynamite were necessary to start the caisson moving. Steel drums 7 ft. in diameter were placed around the wells, and the caisson was loaded with mud for added weight to aid sinking. At Elevation —37 the west end of the attached coffer-dam was seen to be pulling free from the caisson and sinking was stopped. At this point the top of the pier base was at Elevation +6 and the ground surface at Elevation +41, which gave a vertical height of 35 ft. of attached coffer-dam subject to skin friction. Thirty days were lost while 45-ft. steel sheet-piling was secured and driven around the caisson. This sheeting was driven quite close to the attached coffer-dam and penetrated to Elevation —4, which was 4 ft. below the final elevation of the top of the pier base. Sinking was resumed inside the steel-pile protection, and the caisson landed at Elevation —42.8, ⅞ in. east of line and 1½ in. north, with 3⅞-in. skew. Borings were made to investigate the bottom material and it was found to be satisfactory. The caisson was sealed "in the dry," the material being placed along the cutting-edge by hand and, finally, dropped from the top of the wells to insure the proper filling of the working chamber.

Abutments.—Piers Nos. 1 and 4 were excavated to a depth of 40 ft. by hand. The material was loaded into buckets which were handled by the stiff-leg derrick. An open timber coffer-dam was used and at Elevation +40 considerable water was encountered. The foundation was excavated a foot lower than grade and a mat of dry concrete was placed to assist in controlling the water and to provide proper material on which to pour the reinforced footing. A sump for pumping was left, and when the footing had been placed the sump was plugged. The shafts were completed next. Owing to the appreciable quantity of ground-water present, it was thought advisable to provide some means of handling it, should it tend to uncover the footing. To this end a wall 4 ft. high was poured between the piers and beyond to the end of the footing, with drain pipes placed so that a connection might be made to them. Thus, the water might be carried away without the possibility of erosion close to the pier.

Erection of Steel Work.—The erection plan called for the use of falsework bents under the entire structure, the lift span being erected in the down position. This plan left a clear height of 56 ft. through the then existing canal channel which passed under the south approach span. One bent of falsework was omitted under the fifth panel, leaving a clear width of 25 ft. between bents. Timber fenders were built to protect the pile bents in the narrow channel.

The steel and machinery were delivered by rail at Mount Pleasant, Del., two miles from the bridge site, by a concrete road, and were trucked to the job where the traveler unloaded and sorted them in the yard.

A 10-ton traveler with 50-ft. boom was used to set the falsework as well as the steel in the floor system and the trusses of the three spans. When the spans were completed a guy derrick was set on falsework on top of the north end of the lift span and by the use of a 100-ft. boom all the north tower steel was placed. The derrick was shifted from the north to the south end of the lift span in 8 days, and the south tower was erected in 14 days thereafter. The cost of this bridge was $475 000.

SUBSTRUCTURE OF REEDY POINT BRIDGE

The Reedy Point Bridge was built over the already dredged, but not then used, new entrance south of Delaware City. The abutment and pier details were quite similar to those at St. Georges, except that timber piles were used under the abutments instead of concrete. (See Fig. 9.) The site was level and entirely free from obstruction of any nature. Abutment excavation was in a light sand and jetting of the piles was not necessary; 26-ft. piles reached the required bearing value of 20 tons.

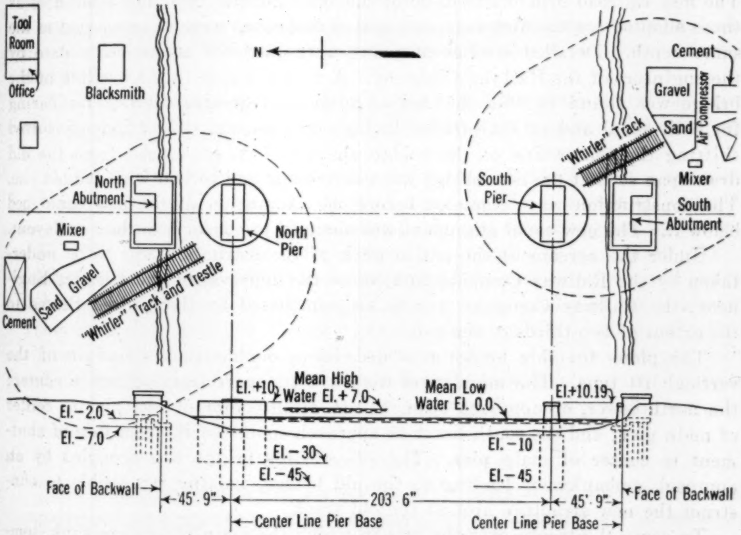

Tool Room
Office
Blacksmith
N
Cement
Gravel
Sand
Air Compressor
"Whirler" Track
Mixer
South Pier
South Abutment
North Abutment
Mixer
North Pier
Cement
Sand
Gravel
"Whirler" Track and Trestle
El. +10
Mean High Water El. + 7.0
Mean Low Water El. 0.0
El. +10.19
El. − 2.0
El. − 7.0
El. − 30
El. − 45
El. − 10
El. − 45
Face of Backwall
45' 9"
203' 6"
45' 9"
Face of Backwall
Center Line Pier Base
Center Line Pier Base

FIG. 9.—PLANT LAYOUT, REEDY POINT SUBSTRUCTURE.

Caissons.—In order to avoid the building of floating caissons, circular coffer-dams, 43 ft. in diameter, were driven in 40-ft. lengths of 13½-in., straight-web steel sheet-piling to Elevation —30 and filled up to above high water with sand dredged from the canal. The caissons were built on these islands. Excavation was done partly by orange-peel bucket and partly by hand.

Erection of Steel Work.—The shore lines, at high tide, were against the abutment faces; the channel was closed to all traffic and the Government loaned the contractor a 10-ton derrick-boat; these last named features permitted simple falsework erection (3-pile bent) and the building of the lift span complete, including concreting and paving, in the down position.

Some steel was delivered by barge from the rail-head at Delaware City, but mostly it was trucked from Wilmington, Del., and unloaded by the traveler brought from Summit for the purpose.

The bottom sections of the towers were erected by the derrick-boat on the south side and by the traveler on the north side. The same creeper that was used at Chesapeake City, revised for the narrower width, was used to erect the remaining tower steel and no difficulty was encountered. The sheaves and the counterweight ropes were set by means of the creeper before it was finally lowered to the ground. The lift span was erected in part by the derrick-boat and in part by the traveler. The cost of this bridge was $318 000.

Bridge of the Pennsylvania Railroad Company

By agreement with the Chief of Engineers, United States Army, the Pennsylvania Railroad Company arranged to rebuild its bridge across the canal. The new railroad bridge was to be of the same general type and clearances as those adopted for the highways, and pier foundations were to be carried to the same depth. Detailed studies of plans were initiated at an early date by the engineers of the Railway Company. A slight change in the location of the bridge was found to offer decided economy and greater facility for caring for both canal and railway traffic during construction. The change involved shifting the center line of the bridge about 250 ft. southward from the old draw-span so that the new bridge was constructed end to end with the old one. The construction was completed before the excavation of the new canal bed below it. The new canal alignment was changed to conform to the new layout.

Under the agreement the entire work of reconstruction was to be undertaken by the Railway Company subject to the approval of the District Engineer; the Railway Company was to be reimbursed by the United States to the extent of two-thirds of the cost.

The plans for this bridge provided for a double-track structure of the vertical lift type. The main piers were 211 ft. apart from center to center; the north tower, or approach span, 172 ft. from center of rest pier to center of main pier; and the south tower or approach span, 106 ft. from face of abutment to center of main pier. The site of this bridge was occupied by an approach embankment leading to the old bridge, making it possible to construct the new structure around the old track.

To carry the tracks over the pits in which the main piers were sunk, temporary girders, supported on pile abutments, were installed, and switches were provided to shunt all regular railway traffic to the northbound track on the bridge. The southbound track was left free for use of the necessary construction plant engaged on the new work.

In all essential elements, the design of this bridge was the same as that of the highway bridges, except for the heavier loading. The main piers were sunk by the same process and to the same depths as those for the highway bridges. The foundation material was not as satisfactory as could have been desired (especially that in the north main caisson) and, later, gave rise to troublesome pier movements. There was, however, no promise of finding better material by going deeper, so it was necessary to make the best of the material encountered. The share of the cost of this bridge paid by the United States was $552 000.

PIER MOVEMENTS

The St. Georges, Summit, and the Pennsylvania Railroad Bridges were constructed prior to the excavation of the new canal prism past their sites, and subsequent to or during such excavation, movements of the main piers occurred in all three cases. The principal movement in each case occurred on the north pier, and in the latter two cases it was sufficient to interfere with the operation of the bridge. The maximum movement seems to have been about 3 in. horizontally, that is, the piers moved toward the channel that much, while the railroad bridge pier not only moved horizontally about the same amount, but also settled about 3 in. This movement in each case appears to have been due to a slight tipping of the pier due to a re-adjustment of earth pressures after the foundation material had been disturbed by the excavation of such a large prism. It was necessary to resort to corrective measures in each case in order to render the bridge operative without danger to either canal or land traffic.

The railway bridge was raised by jacking, and the north tower was moved back enough to give proper clearances. Shims were placed under it to raise it to proper grade. After this operation it worked satisfactorily.

A different method was used on the St. Georges Bridge, where a horizontal movement alone had occurred. A pump-boat capable of supplying water for seven or eight jets of 1½-in. pipe was available, and jets were sunk on the land side of the piers until the material was loosened all the way down. At the same time a 3-in. steel cable was passed around the pier in a loop, and the two ends were brought back to substantial "dead men" located near the abutments. These ends were attached to screw-jacks working on the "dead men" to give a strain on the cables. The cables were tightened as the jetting proceeded, and, in this manner, the pier was slowly brought back into its proper position. The same method, with suitable variations, was then successfully applied to the Summit Bridge.

ECONOMIC VALUE OF CANAL

The soundness of a prediction for an early demand for an increase in the capacity of this waterway has been questioned. Antagonists of waterway improvements have declared it a waste of public funds. This is not the case in the instance of the Chesapeake and Delaware Canal. Assuming that a public work like this is for public good, for public convenience, and for public economy, this canal makes an excellent showing for the funds expended on it. Omitting all consideration of pleasure boats, passengers, and empty vessels, it gives passage to from 600 000 to 700 000 tons of freight annually. Knowing the classification of this freight, its points of origin and destination, and the proper carriers rates, it is possible to compute the actual cost of transporting that freight by water, and to compute what it would have cost if shipped by rail, thus giving the direct saving, if any, due to the existence of the canal.

This computation was made in 1926 for the canal for the entire period since it was taken over by the United States in 1919. From the annual savings thus computed there was deducted all actual operating costs of the canal, an assumed sinking fund of 2% to amortize the total cost in fifty years, and interest at 4% on all appropriations. After making these deductions, the net savings in freight transportation costs (which is thus added to the National wealth) is in round numbers $1 000 000 per year, or, say, $1.25 for every ton of freight passing through the canal.

HYDRAULICS

Prior to the completion of the canal, considerable interest from technical and practical standpoints attached to the question of the tides and currents that might be expected after the completion of the projects. The question involved a study of the result of the interference of two unequal opposed waves in a canal of limited length and of practically constant cross-section. There is little knowledge in the engineering world of the resultant phenomena in such cases. Navigators are interested in them from a practical standpoint, since they will have a bearing on the passage of vessels through the canal. Early attention, therefore, was given to studies of phenomena to be expected with a view to predicting them beforehand, and subsequently checking the predicted results by observing the actual phenomena.

The canal was to be 90 ft. wide on the bottom, with side slopes of 1 on 2. It was to be somewhat wider at the two ends, but this extra width was neglected in studying the problem. The depth was to be 12 ft. at mean low water, referred to the Sandy Hook datum plane; that is, its bottom was to be level. It has only a few curves, which are of long radius, and these were also neglected. Its length from Delaware River to Back Creek is 13.5 miles, and it extends down Back Creek (a slowly widening stream) for 4.5 miles. Back Creek was omitted from consideration in the computations for the sake of simplicity. This omission undoubtedly involved some error, but how much is not known. It is thought to be small. The major tide entering the canal (an average amplitude of 6 ft.) comes from the Delaware River. The minor tide entering from Chesapeake Bay has an average amplitude of 2.4 ft. at Chesapeake City. Mean tide level is at substantially the same elevation at each end. The smaller tide reaches Chesapeake City two hours earlier than the major tide passes the jetties in Delaware River. The bottom of the canal is 15 ft. below mean sea level.

There are many localities where unequal and opposed tides meet in natural channels, but in most such cases the phenomena peculiar to this problem are so masked by others due to purely local conditions (such as varying widths and depths) that very little progress has been made in their study, and but few studies are available in the English language.

Canals of fairly constant depth and cross-sectional area are usually artificial; and it is to this class that engineers must look principally for a solution of the problem proposed.

The Cape Cod Canal, which is an artificial cut connecting Cape Cod Bay with Buzzards Bay, in Massachusetts, presents the problem in a manner very similar to the Chesapeake and Delaware Canal. It has been described by William Barclay Parsons,[5] M. Am. Soc. C. E., who based his study on the writings of Airy, de St. Venant, and Maurice Levy. The equations he gives are substantially those of Levy.

Probably the best study of the problem in a natural channel is that of the English Channel by M. Bourdelles,[6] who does not attempt to solve the problem mathematically; but his explanation of the theory is excellent. By following the theory of M. Levy as given by Mr. Parsons, the writer has computed the tides and currents to be expected in the Chesapeake and Delaware Canal. The predicted tides as given by this computation are shown graphically in Fig. 10 (*a*), and the predicted currents are shown in Fig. 10 (*b*).

A study of the assumptions made in the deduction of the Levy equations, however, led the writer to doubt their validity, especially in the case of such a shallow canal as the one under consideration. He concluded that: (1) The wave propagated from each end should be considered merely as a wave of translation, with a positive phase during high water and a negative phase during low water; (2) its height at any point in the canal should vary sinusoidally as did the generating waves in the respective bays; (3) each of these waves of translation contained a certain quantity of energy; and (4) one-half the energy was potential and was represented by the height of the wave above mean sea level, and the other half was kinetic, represented by the velocity of the water at a given point; (5) this energy was expended in overcoming friction in accordance with a law which was deduced, resulting in a consequent falling off of height of wave and of velocity as the wave was propagated farther into the canal; (6) the resultant height at any point in the canal at any instant was the algebraic sum of the instantaneous heights due to each wave at that point; and, (7) the resultant velocity at any point was the algebraic difference between the instantaneous velocities due to each wave at that point. Fig. 11 shows the resultant heights and velocities as computed under these assumptions.

There are no striking differences in the results. Both indicate that (*a*) tidal propagation is not instantaneous; (*b*) in general, the canal surface between the two ends will be concave on a rising tide and convex on a falling tide; (*c*) the larger tidal oscillation at the Delaware River progressively decreases in amplitude to a point about 2½ miles from Chesapeake City; (*d*) the resultant wave is propagated in the direction of west to east, or against the direction of propagation of the major component wave; and (*e*) the speed, or rate of travel of the wave, is very small at the western end but extremely rapid at the eastern end.

Some discrepancies are found between the two results. The Levy formula gives symmetrical curves for high and low water, while the wave-of-translation formula does not. It gives amplitudes of greater magnitude for the positive

---

[5] "The Cape Cod Canal," by William Barclay Parsons, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1.

[6] "Etude sur le Régime de la Marée dans la Manche," Bourdelles, *Annales des Ponts et Chaussees*, 1899, 3^{me} Trimestre.

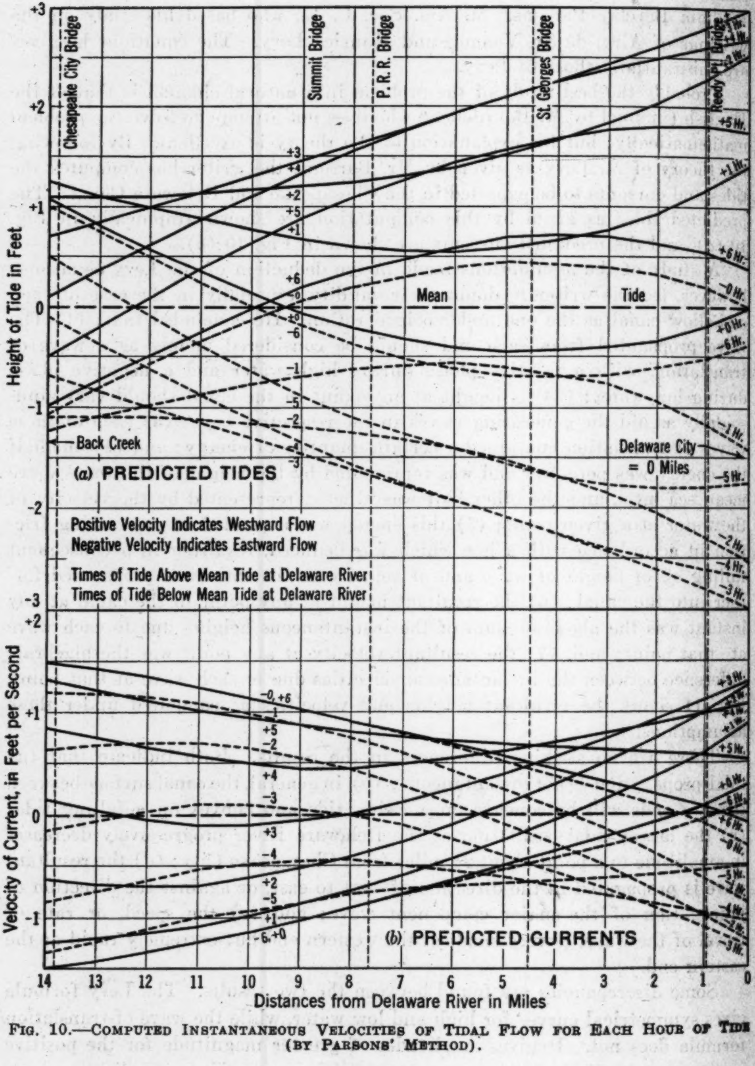


FIG. 10.—COMPUTED INSTANTANEOUS VELOCITIES OF TIDAL FLOW FOR EACH HOUR OF TIDE (BY PARSONS' METHOD).

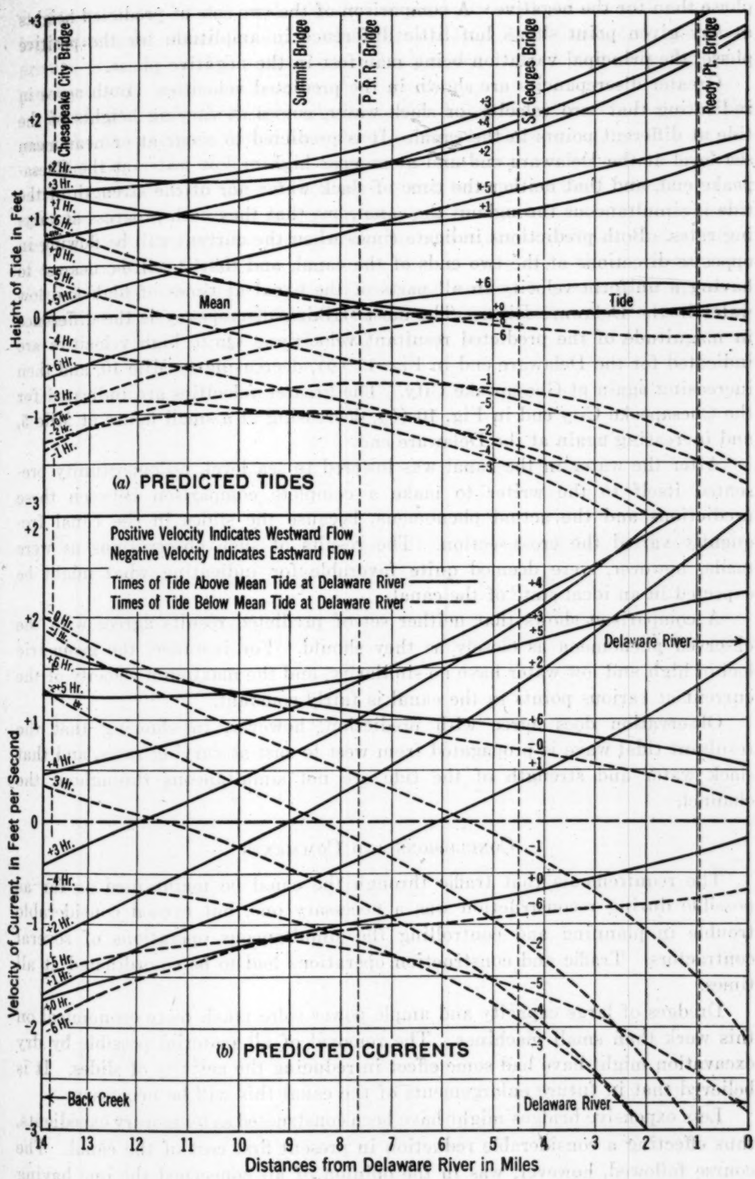


FIG. 11.—INSTANTANEOUS VELOCITIES OF TIDAL FLOW COMPUTED BY THE WAVE-OF-TRANSLATION FORMULA.

phase than for the negative. A comparison of the two sets of predicted heights at any given point shows but little difference in amplitude for the positive phase, the principal variation being manifest in the negative phase.

Greater discrepancies are shown in the predicted velocities. Both agree in indicating that zero velocity, or slack water, comes at varying heights of the tide at different points in the canal. It is predicted to occur at or near mean sea level at the Delaware end and at or near high or low water at the Chesapeake end, and that neither the time of slack water nor of the strength of the tide is simultaneous throughout the canal, but that they both progress at varying rates. Both predictions indicate times when the current will be flowing in opposite directions at the two ends of the canal, and that it will be nearest to having a uniform velocity in all parts of the canal at times of high and low water in the Delaware River. The most striking discrepancy is the difference in magnitude of the predicted resultant velocities. Quite high velocities are indicated for the Delaware end in Fig. 11 (*b*), decreasing to Mile 10, and then increasing again at Chesapeake City. The greater velocities are indicated for the Chesapeake City end in Fig. 10 (*b*), decreasing at a small figure at Mile 5, and increasing again at the Delaware end.

After the water in the canal was lowered to sea level, no opportunity presented itself to the writer to make a complete comparison between these predictions and the actual phenomena, because the slides in the canal frequently varied the cross-section. The results of such observations as were made, however, were deemed quite favorable for indicating what might be expected in an ideal state of the canal.

A comparison shows that neither set of predicted results agrees with the observed phenomena as closely as they should. For instance, the geometric loci of high and low water have no similarity, and the maximum velocity of the current at various points in the canal is fairly constant.

Observation does agree with prediction, however, in showing that the resultant tidal wave is propagated from west to east at varying rates, and that slack water and strength of the tide are not simultaneous throughout the channel.

Conclusions and Comments

The requirements that traffic through the canal be maintained as far as possible during reconstruction was a necessary one, but caused considerable trouble in planning and controlling the simultaneous operations of several contractors. Traffic and construction operations had to be co-ordinated at all times.

Dredges of large capacity and ample power were much more economical on this work than small machines. The removal of all material possible by dry excavation might have had some effect in reducing the activity of slides. It is believed that in future enlargements of the canal this will be necessary.

Less expensive bridges might have been constructed as temporary expedients, thus effecting a considerable reduction in present first cost of the canal. The course followed, however, was in the opinion of all concerned the one having the greater ultimate economy.

The provision of an ample right of way for the disposal of spoil is exceedingly desirable where land is relatively cheap. Any damage to adjacent private property leads to vexatious disputes, exorbitant demands for compensation, and possible litigation.

Wet borings are absolutely unreliable for determining the character of the sub-soil. Dependence should be placed on core borings only.

Tidal variations of level and currents in the canal are on the whole detrimental to navigation. The detriment lies not so much in their effect on speed and time of transit, as on safety to the vessel. If a large and heavily laden vessel grounds and gets athwart the channel, as it may readily do under action of currents, a fall in the tide may break the vessel in two. Tows approaching closed bridges with a fair wind and tide, and having to check their speed on account of failure of the bridge to open, are particularly exposed to having one or more vessels of the tow ground.

DISCUSSION

F. T. CHAMBERS,[7] M. AM. SOC. C. E. (by letter).—In recording the construction stages of the Chesapeake and Delaware Canal and in giving quantities, yardage, costs, and methods in connection with the dredging operations of the conversion from lock-level to sea-level, the author has done a useful service. The description of the deposit of spoil in the high-level disposal areas by means of modern hydraulic dredges of standard construction and the record of costs of this part of the work are in themselves sufficient justification of the paper without its many other features. The dredge performance is the more creditable by reason of the condition of the material as encountered.

While the history given is sufficient as to the physical aspects and the Canal Company finances, a bibliography of all publications concerning this canal and the many projects in general for canals across the Delaware Peninsula should be attached to this paper. Several public documents treat of this subject. A single report[8] relates much of this effort and records other reports with borings along this and other routes.

Public Joint Resolution No. 37, approved June 28, 1906, authorized the President of the United States to appoint a commission consisting of an officer of the Engineer Corps of the Army, an officer of the Navy, and one person from civil life,

> "* * * to examine and appraise the value of the works and franchises of the Chesapeake and Delaware Canal, connecting the waters of the Chesapeake and Delaware bays, with reference to the desirability of purchasing the said canal by the United States and the construction over the route of the said canal of a free and open waterway having a depth and capacity sufficient to accommodate the largest vessel afloat at mean low water;" and, also, "to the extent that the same can be done from the surveys heretofore made under the direction of the War Department and within the limits of the appropriation herein made" to "examine and investigate the feasibility, for the purpose of such a waterway, of the route known as the Sassafras route."

The Commission was ordered to:

> "Make a report of its work, together with its conclusions upon the probable cost and the commercial advantages and the military and naval uses of each of the said routes, to the Secretary of War, who shall transmit the same to Congress at its next session."

The late Gen. Felix Agnus, of Baltimore, Md., was the Chairman and Maj. (late Maj. Gen.) C. A. F. Flagler, Corps of Engineers, U. S. A., M. Am. Soc. C. E., and the writer were the other members of that Commission. From the very nature of its precept, the Agnus Commission was obliged to use past reports and other available data, and thus Senate Document No. 215 contains, in addition to survey and boring figures resulting from the field work of its own personnel, much that appears in previous reports.

Although it was the apparent belief of many interested people that the Chesapeake and Delaware Canal should be taken over by the National Government and converted to sea-level operation by stages as the traffic demand might require, it will be seen from the text of the Congressional Resolution that the Agnus Commission started with a considerable handicap to full investigation. It was limited not only in time and money, but more particularly in the requirement of a canal to accommodate the largest vessel afloat. Although the Commission took considerable liberty in its interpretation of that expression it was still obliged to consider a depth of 35 ft. at mean low water. True, it took further liberties and figured a 30-ft. depth also but, because of its instructions, it could not estimate on barge depths. It did, however, go so far as to recommend the purchase of the Chesapeake and Delaware Canal for a sum not to exceed $2 514 289.70 and to record the public pressure for gradual deepening. It may be stated here that the Commission assumed that much of the high-elevation excavation would be steam-shovel work.

---

[7] Rear-Admiral (C. E. C.), U. S. N.; Director, U. S. Naval Petroleum Reserves and Oil Shale Reserves, Washington, D. C.

[8] Senate Doc. No. 215, 59th Cong., 2d Sess.

Although the Commission reported upon the Sassafras route, it found in favor of the Chesapeake and Delaware route, recommending a sea-level canal following much the same lines as that described by the author, but debouching upon the Delaware several thousand feet below Reedy Point. It also found for jetties at the Delaware entrance "for at least 1 000 ft. from shore".

The author's study of the canal hydraulics is interesting. It is to be hoped that observations of actual tidal heights and currents will be made and published. The Angus Commission gave consideration to the matter of currents and to a difference of tidal heights between the Chesapeake and the Delaware ends of 6 to 10 ft. It also gave consideration to the tidal lag at the Chesapeake end. It compared the physical conditions at the Chesapeake and Delaware Canal with those at Suez, where M. Quellenec reported an extreme tidal difference of 9.02 ft. and a resultant current of 2.6 knots. Furthermore, it cited the lockless sea-level canals of Corinth and Tasmania, which are easily traversed by vessels, together with computations made for a tide-level Panama Canal. It did not attempt, on account of its limited time and funds, to make complex computations based on any recognized formula, but it did make rough over-all estimates. It concluded that a tide lock was not indicated as part of the cost.

The writer has nothing to criticize in this able paper. He agrees with the author in the conclusion that in future enlargement of the canal it may be a saving of expense in the lessening of landslides to remove some part of the overburden by dry methods.

His only disagreement is with regard to wet borings. Such borings are not absolutely unreliable for determining the character of the subsoil. The writer has made or supervised the making, first to last, of many borings at numerous places. He does not choose wash-rod borings where time and money will procure borings of a more positive type. A small percentage of his borings have been exclusively wash borings. Many wash borings are worthless, or nearly so; but the wash rod with casing can be applied with considerable satisfaction for the determination of some subsoil conditions, especially of conditions for dredging and where due care is had in the use of sampling devices, ancillary to the methods of the wash rod.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1778

EFFECT OF TURBULENCE ON THE REGISTRATION OF CURRENT METERS[1]

BY DAVID L. YARNELL[2] AND FLOYD A. NAGLER,[3] MEMBERS, AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. CHARLES S. BENNETT, CARL ROHWER, BREHON B. SOMERVELL, N. C. GROVER, BURKE L. BIGWOOD, G. H. NETTLETON, C. LINDEN, H. R. LEACH, LUDWIG A. OTT, B. F. GROAT, FORREST NAGLER, ROBERT E. HORTON, RICHARD VAN VLIET, AND DAVID L. YARNELL AND FLOYD A. NAGLER.

SYNOPSIS

This paper describes a series of experiments which were conducted on several types of current meters for the purpose of determining the effect of turbulence, or variations in direction, and rapid changes in speed of flow, on their registration. In contrast with experiments hitherto made by moving the meter in still water, these tests were made by holding the meter stationary in a controlled stream of water.

The results of the tests are shown in diagrams and the conclusions are given at the end of the paper.

INTRODUCTION

The current meter is an instrument commonly used to obtain water velocities as a part of the process of making discharge measurements on natural streams, and is also frequently used similarly in open artificial channels. The meter is usually rated or calibrated by pulling it at definite speeds through quiet water. Although such a rating may be performed with extreme accuracy, it is obvious that the stream-line condition is generally quite different from the more or less turbulent flow encountered by the meter when held stationary in a moving current. In fact, parallel filaments of flow, such as are duplicated in ordinary rating methods, are seldom seen in water

---

[1] Published in December, 1929, *Proceedings*.

[2] Senior Drainage Engr., Bureau of Public Roads, U. S. Dept. of Agri., Iowa City, Iowa.

[3] Prof. of Hydr. Eng., Univ. of Iowa, Iowa City, Iowa.

flowing in either natural or artificial channels. On the contrary, the main stream current often appears to be the resultant of numerous cross-currents, rotating vortices, and eddies. This is particularly true in natural channels. Less turbulent conditions exist in canals, and gauging stations may usually be located at the lower ends of long tangents where the degree of turbulence is a minimum. Current-meter measurements demanding maximum accuracy often must be made in the head and tail-races of power plants where there is likely to be considerable turbulence.

Eddies encountered in flowing streams may be considered as circling or whirling masses of water moving within the whole mass of the stream. In other words, each eddy is, in effect, a separate body of water having a momentum of its own entirely different both in direction and magnitude from that of the main current. When the impact of such a body is transmitted to the moving wheel of a current meter, its registration, for the moment, will be different from that obtained in the uniformly flowing stream.

With these eddies passing irregularly in both time and magnitude, individual velocity measurements made at a given point by a current meter may be subject to considerable variation for no other reason than that the meter was not held long enough at the point to obtain an integrated mean representing the true average current. Believing that such departures may be of a compensating nature in combining all the observed points in a given stream measurement, engineers seldom spend more than 90 sec. in securing an individual reading. Nevertheless, even in mildly eddying water, considerable variation will be obtained in two consecutive readings of the ordinary duration, and the possible departure increases with increasing turbulence.

Current meters are made in various styles, but all have a rotating element, called the wheel or propeller, actuated by the moving stream. The instrument is generally designed so that the moving wheel forms, through the proper mechanism, a contact in a compartment where the electrical terminals for registering its revolutions are placed. Thus, the number of revolutions of the wheel may be obtained by connecting the meter with an electric counter, bell, or buzzer. Some meters are constructed so that the contact is made after the wheel has turned a certain number of revolutions as 1, 5, 10, 25, or 50. Thus, from the speed of rotation as measured over a definite period of time, the velocity of the flowing water is determined.

The current meters in common use may be classified as of two types: The cup type in which the wheel rotates about a vertical axis; and the screw type in which the wheel or propeller rotates about a horizontal axis, ordinarily intended to be held parallel to the direction of flow of the stream. When a meter is held at a definite point in a turbulent stream, it may be acted upon during successive instants by different stream filaments which may be flowing in various directions and may have various speeds. Even if the current meter is held rigidly at one place, the wheel will then be acted upon, not only by stream filaments which strike the rotating element at various angles, but also by filaments with appreciable variations in speed. The combination will cause the speed of the wheel to be alternately accelerated and retarded. There has existed hitherto a dearth of information regarding the effect of turbulence in flow on the registration of different types of current meters, and it was to throw light on this question that the present investigation was made.

Some experimental studies have been made in the past to determine the characteristics of different types of meters under circumstances departing from the good velocity conditions usually found at most gauging stations; but hitherto few data have been published showing the specific deviation to be expected under abnormal conditions in using either of the common types of meter. Nearly all canal gauging stations and many river stations have conditions that yield readings to an accuracy that is within 1 to 4 per cent. Exceptions to these stations occur where gaugings must be made: (1) From a bridge with pier obstructions disturbing the flow; (2) in mountain streams and canals with bottoms strewn with rocks of various sizes; (3) on streams in which whirls and boils with mud and sand travel slowly with the stream; and (4) where gaugings must be made along narrow crests that develop vertical angularity to the flow. At such stations the hydrographer desires particularly a meter that will be actuated by the forward components of velocity only, provided the meter equipment is such that he can hold the instrument directly up stream. If they are free to turn and face the local current, all meters, of course, will reflect the direct velocities which are greater than the down-stream components perpendicular to the gauging section. The ideal situation, with parallel filaments of flow, would usually involve velocities too low to turn a meter with acceptable accuracy.

A summary of the comments of American experimenters in this field is given in the Appendix. The experience of these investigators seems to indicate that, under normal conditions such as exist at the majority of gauging stations, the current meter is to be regarded as a reliable instrument for measuring water velocity. Obviously, the best available conditions for stream gauging should always be sought. When eddies do exist, some meters have been found to register more, and others less, than the true forward component of the turbulent water. This forward component is equal to $V \cos \alpha$, in which, V is the velocity of the water, and α, the angle between the direction of local flow and the axis of the stream. Hereafter, this component will be referred to as the "cosine component".

When these experiments on current meters were begun it was expected that it would be a simple matter to put a given meter into a smooth, straight, uniform channel, carrying a known, constant, parallel-filament flow, and observe the meter reading; then produce violent turbulence by artificial means and measure the changes produced in the meter readings. Upon making the trial, however, it was found impossible to obtain consistent results that could be interpreted to any useful purpose. Therefore, it became necessary to change and expand the entire plan of experimental procedure. One step led to another until finally experiments which, at the outset, it had been intended to complete in 2 or 3 weeks, were carried on at intervals for 1½ years.

The experiments were made at the Hydraulic Laboratory of the State University of Iowa, during December, 1924, May and June, 1925, and May, 1926, by D. L. Yarnell, M. Am. Soc. C. E., Senior Drainage Engineer, under the direction of S. H. McCrory, M. Am. Soc. C. E., Chief, Division of Agricultural Engineering, U. S. Bureau of Public Roads. S. M. Woodward and F. A. Nagler, Members, Am. Soc. C. E., of the State University of Iowa, acted as advisers, making suggestions during the conduct of the experiments and collaborating in the preparation of the data and the report.

TYPES OF CURRENT METERS USED IN THE EXPERIMENTS

Tests were made on the eleven current meters illustrated in Figs. 1 to 8, inclusive. Several of the meters have adjustable contact points so that the meter may record various numbers of revolutions per contact. In these tests, with certain exceptions that are noted where used, the maximum number of revolutions per contact was used for each meter.

It should be noted that the Hoff meter has a rubber propeller. Since rubber, with a specific gravity of 1.3, is but little heavier than water, presumably this meter has less bearing friction and responds more readily to changes in the velocity of water than the all-metel meters. Obviously, also, the flexible rubber propeller is unlikely to be injured by contact with floating débris that might break or bend the metal propeller.

DESCRIPTION OF EXPERIMENTAL INSTALLATION

The water used in the current-meter experiments was drawn from the Iowa River above the University Dam into the Laboratory Testing Canal which is 190 ft. long, and 10 ft. wide. In the canal, below a water-tight bulkhead, was constructed a smooth concrete box-flume (see Fig. 9), 4 ft. wide, 4 ft. deep, and 36 ft. long, in which all the experiments were made. The meters were held by rigid rod suspension in the water, 10 ft. up stream from the lower end of the flume. The entrance to the flume was rounded with a 6-in. radius. For all the experiments the water flowing in the flume was kept exactly 3 ft. deep so that the cross-section of the stream was 12 sq. ft. The constant depth was maintained by means of a check-gate across the canal below the flume. This gate was a loosely fitting rectangular panel, hinged at the bottom, so that its crest could be easily adjusted to the height necessary to maintain the water surface above at exactly the desired elevation.

All the water flowing down the canal passed through the flume and was measured by a 10-ft. suppressed weir near the lower end of the canal. The weir was 8.03 ft. high and had been constructed with great care to secure maximum precision for other laboratory research, during which the computed discharge was checked by volumetric measurements of the flow. Since the cross-section of the canal above the flume was several times as great as that of the water in the flume, the velocity of flow in the canal was low, and, in effect, the flume was drawing water from a quiet pond. The smooth rounded entrance of the flume thus produced a stream of remarkably uniform velocity, much like a jet flowing from an orifice. When there was no obstruction in the flume, measurements with a current meter at eight different points in the cross-section showed almost no measurable variation in velocity. The locations of the points and the measured values of the velocity are shown in Fig. 10.

The quantity of water entering the canal from the river was controlled by a large head-gate. Just below the head-gate is a small submerged sluice-gate which may be opened to allow water to escape from the canal back into the river just below the dam. This sluice-gate was used to make the finer adjustments in regulating the flow through the canal and flume.

Constancy of Experimental Conditions

Experiments were made with velocities of 1, 2, 3, and 5 ft. per sec. in the experimental flume. When, for example, it was desired to run a series at 2 ft. per sec., the head-gate would be opened and the relief sluice-gate would be adjusted until 24 cu. ft. per sec. was flowing over the standard weir at the lower end of the canal. Then, the hinged gate below the flume would be raised until the depth in the flume was exactly 3 ft., thus producing a velocity of exactly 2 ft. per sec. Throughout the experiments the river fluctuated but slowly, so that, when the head-gate had been once set, it was easy during any experiment to keep the variation in quantity of water flowing to a small fraction of 1 per cent.

Since all the meters tested had not been rated recently, and since the study was concerned with the change in the reading of the meter for a certain definite change of conditions and not the absolute value of the meter readings, it was decided at the beginning of the investigation to compare each meter with itself only; that is, in each test the meter reading under what might be called normal or standard conditions was compared with the reading of the same meter when some abnormality of condition had been introduced. Under this plan the investigation could have been confined to one type of meter, or it could have included any number.

At the end of each day it was necessary to close the big head-gate. This gate could not easily be set with precision, the range of regulation by means of the sluice-gate was limited, and the storage in the canal was so large that after the head-gate was opened in the morning considerable time was required for the hook-gauge at the measuring weir to come to a steady constant value. This made it desirable to conserve time. Therefore, an error of 1 or 2% in any particular quantity of flow was considered close enough, and the first setting was held constant throughout the series of experiments with the meter under test. Consequently, where results are shown, for example, for a velocity of 2 ft. per sec., the figures given for one meter may have been obtained with an actual velocity flow of 1 or 2% less than 2 ft. per sec. and with another meter with an actual velocity of 1 or 2% greater than 2 ft. per sec. This procedure was considered justifiable since, whatever actual velocity of flow obtained, it was maintained constant throughout all the experiments on one meter, and the readings obtained with one meter were compared with themselves only and not with the readings of any other meter.

Effect of Turbulence in Producing Errors in Current-Meter Registration

In the first or preliminary series of experiments, in a flume carrying a constant quantity of water, varying degrees of turbulence were purposely

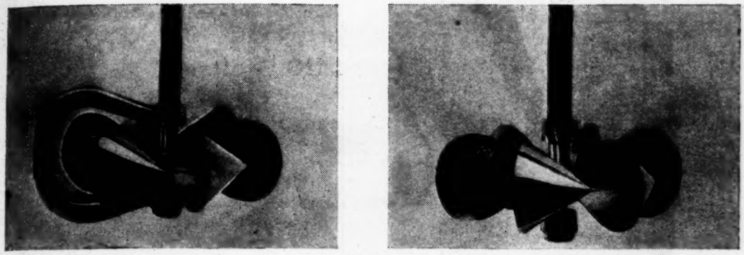


FIG. 1.—PRICE ACOUSTIC METER.

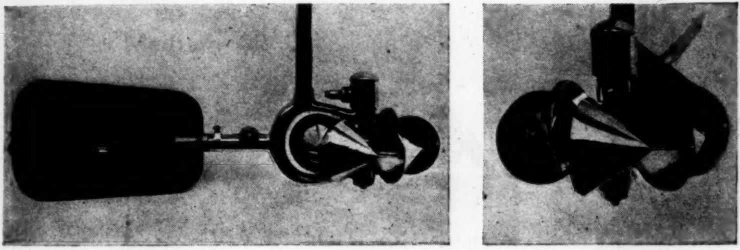


FIG. 2.—SMALL PRICE ELECTRIC METER.

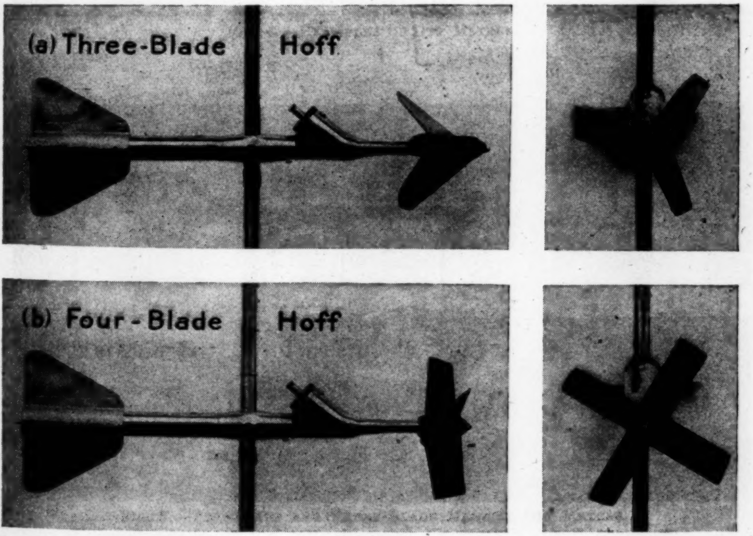

(a) Three-Blade Hoff
(b) Four-Blade Hoff

FIG. 3.—EXAMPLES OF HOFF METER WITH RUBBER PROPELLERS.

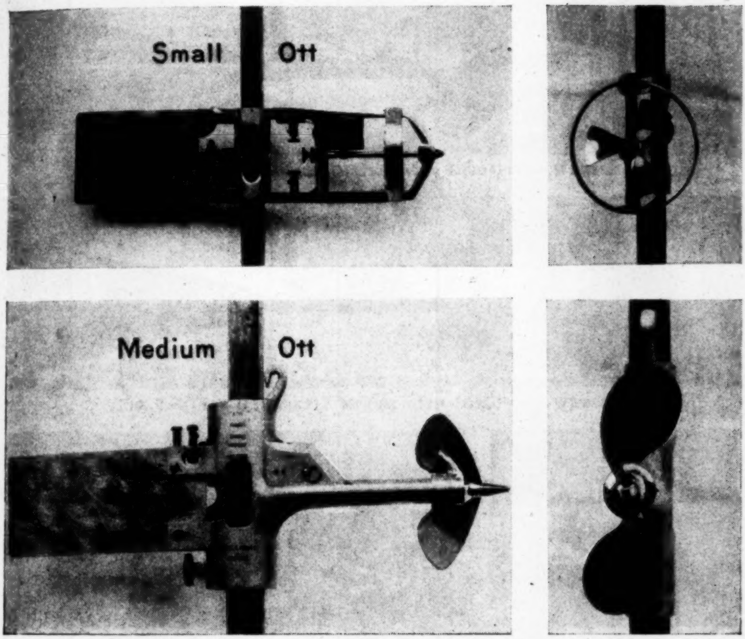

Small Ott
Medium Ott

FIG. 4.—EXAMPLES OF OTT METERS.

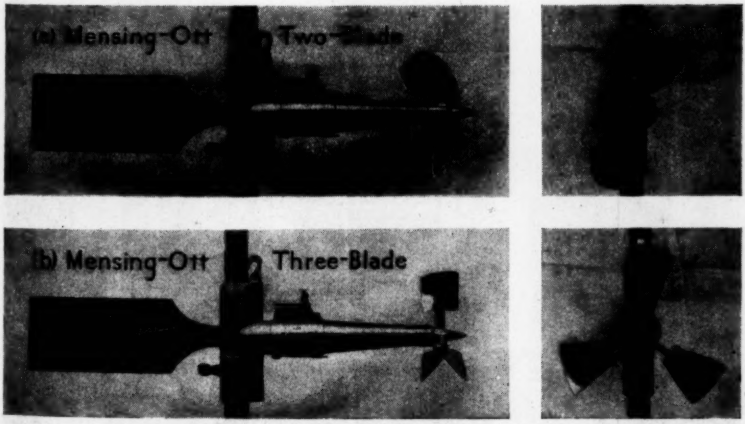

(a) Mensing-Ott Two-Blade
(b) Mensing-Ott Three-Blade

FIG. 5.—TWO-BLADE AND THREE-BLADE MENSING-OTT METERS.

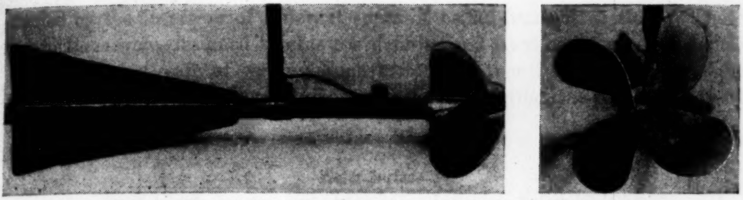


FIG. 6.—TYPE B, HASKELL METER, WITH HIGH-PITCH WHEEL.

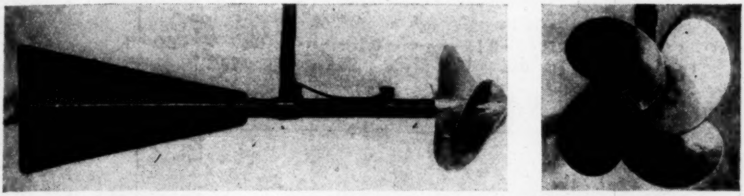


FIG. 7.—TYPE B, HASKELL METER, WITH LOW-PITCH WHEEL.

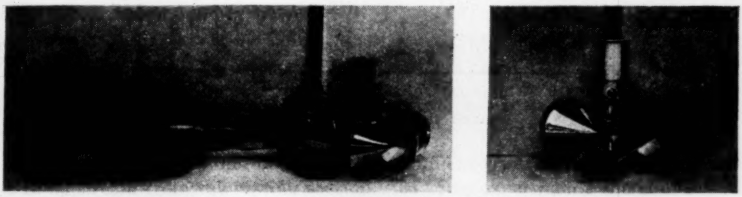


FIG. 8.—U. S. GEOLOGICAL SURVEY TYPE OF IMPROVED PRICE METER.

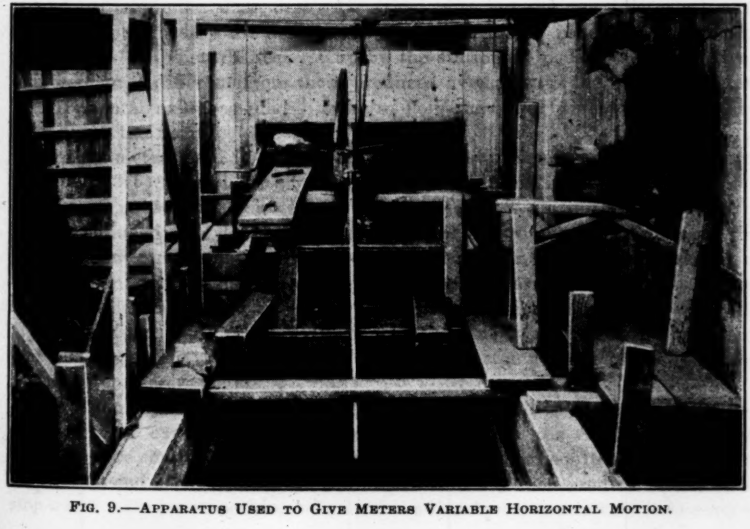


FIG. 9.—APPARATUS USED TO GIVE METERS VARIABLE HORIZONTAL MOTION.

created by the insertion of different types of baffles and obstructions. These obstructions were designed to create a definite and fixed constant disturbance in the flowing stream of water. Each meter was then held in turn at eight fixed points in the flume down stream from the obstruction, the points indicated in Fig. 10.

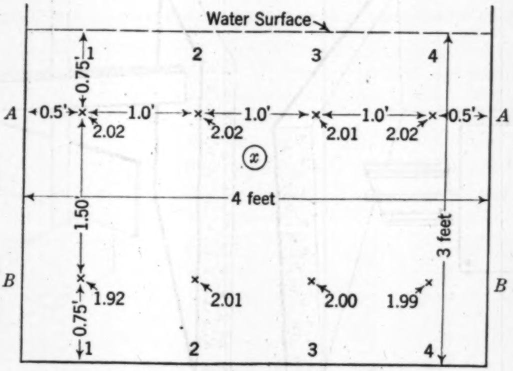


FIG. 10.—CROSS-SECTION OF CHANNEL LOOKING UP STREAM. CROSSES INDICATE METER POSITIONS USED IN EXPERIMENTS THAT FORM FIRST SERIES.

The fixed obstructions creating the turbulence were placed in the flume 6 ft. up stream from the testing section. In some experiments the eddies were produced by either vertical or lateral obstructions in the flume made by a submerged triangular dam, or by a vertical pier, but the most disturbed condition was obtained by the devices resembling stationary paddle-wheels, shown in Fig. 11. The disturbances set up by the stationary paddle with a vertical axis were quite different from those produced by that with the horizontal axis. The horizontal axis was placed so as to hold the lower edges of the paddles 3 in. above the floor.

The six meters used in this preliminary series of experiments are shown in Figs. 1, 2, 3(*b*), 4, and 5(*b*). Eight definite locations in the testing section shown in Fig. 10 were selected for the readings. Tests were run first with no obstructions in the flume. The testing flume had rounded entrance corners, and the flow conditions were more nearly ideal than ordinarily would have been encountered in natural streams, or even in artificial channels. The tests were then duplicated with the fixed obstructions in the flume.

The uniformity of velocity distribution with no obstruction in the flume may be seen in Fig. 10. The velocities shown are the average of the readings taken by four screw meters.

Since the object of this series of experiments was to determine what variation might be expected in the actual use of meters, the revolutions of the propeller were counted for a period no longer than is commonly used (generally between 40 and 90 sec.), and the time was accurately measured by means of a stop-watch. Undue smoothing of a local reading by an excessively long period of time was purposely avoided as impractical in field practice. In Experiments Nos. 8 and 9, duplicate readings as shown in Table 1 were taken at each point in the cross-section in order to determine the possibility of error by taking observations for too short a time in turbulent water.

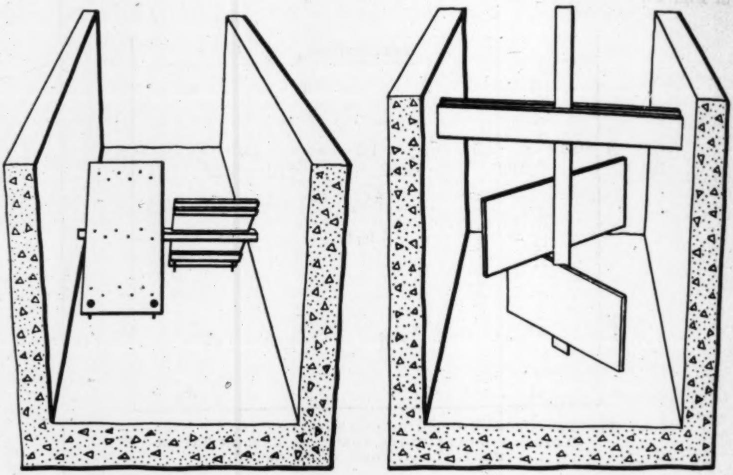


FIG. 11.—STATIONARY PADDLE-WHEELS USED FOR CREATING TURBULENCE.

At the conclusion of Test No. 9, an additional series was run to ascertain whether the meters performed in a radically different manner when the center of the propeller of each meter was located accurately in the plane of the testing section in contrast to the earlier procedure of locating the supporting rods themselves in this section. Although the distance from rod to meter propeller varied somewhat for the different instruments, the relative performance of the meters was altered but little by the change.

The entire series of observations for four tests are presented in Table 1. These include, for a velocity of 2 ft. per sec., experiments with the stationary paddle-like obstructions shown in Fig. 11.

When readings for the various meters are compared for the same point in the channel, it may be seen from Table 1 that in case of turbulent flow there is considerable difference. For example, at Point A_2 in Test No. 8, the Price meters over-registered, whereas the screw meters under-registered. In many cases the registration of the cup type of meter is more than 200% that of the propeller type for certain points in the tests with turbulent flow.

The check observations in Tests No. 8(*a*) and No. 9(*a*) indicate that individual observations of velocity in this turbulent water, taken for the usual length of time, may vary as much as 10 to 20%, but even this difference (which may be compensating in an average of the many observations taken in a discharge measurement), is not nearly so significant as the error of uniform sign due to the continual over-registration or under-registration of the meter in eddies and turbulent water.

While the data of Table 1 do not disclose the correct values of the average velocity of the current, it became evident from the angularity tests made later that the Price meter recorded too high a value and that the values obtained with the other meters are in varying degrees below the true amount.

TABLE 1.—COMPARATIVE CURRENT-METER READINGS SHOWING THE EFFECT OF TURBULENT FLOW ON THE REGISTRATION OF VARIOUS TYPES OF METERS.

Test No.	Point at which meter was held.	Measuring conditions.	VELOCITY, IN FEET PER SECOND.					
			Price Meter.		Hoff meter.	Ott Meter.		
			Electric.	Acoustic.		Small.	Medium.	Large.
	(Fig. 10.)	(Fig. 11.)	(Fig. 2.)	(Fig. 1.)	(Fig. 3.)	(Fig.4(*a*).)	(Fig.4(*b*).)	(Fig. 5.)
8	A_1	Horizontal paddle	2.71	2.68	3.01	2.54	2.71	2.91
	A_2	" "	2.36	2.50	0.97	1.03	0.93	0.81
	A_3	" "	2.49	2.89	1.70	1.43	1.67	1.38
	A_4	" "	2.13	2.41	2.31	2.10	1.90	2.13
	B_1	" "	2.36	2.68	2.53	2.62	2.70	2.86
	B_2	" "	2.41	2.20	1.29	1.40	1.23	1.46
	B_3	" "	1.57	1.62	0.66	0.62	0.30	0.40
	B_4	" "	1.77	1.67	1.58	1.83	1.97	2.31
8(*a*) duplicate of Test No. 8.	A_1	" "	2.99	2.99	2.55	2.70	2.80	2.96
	A_2	" "	2.33	2.61	0.96	1.04	0.91	0.74
	A_3	" "	2.47	2.80	1.78	1.53	1.67	1.46
	A_4	" "	2.04	2.42	2.28	2.17	1.92	2.12
	B_1	" "	2.42	2.48	2.44	2.76	2.43	2.82
	B_2	" "	2.57	2.46	1.40	1.57	1.06	1.15
	B_3	" "	1.59	1.78	0.85	0.71	0.27	0.55
	B_4	" "	1.65	1.55	1.56	2.05	1.57	2.32
9	A_1	Vertical paddle........	2.32	2.58	2.01	1.94	1.81	1.72
	A_2	" "	2.67	2.78	1.36	2.07	1.17	1.10
	A_3	" "	2.23	2.28	1.34	1.69	1.22	1.23
	A_4	" "	2.33	2.40	2.46	2.13	2.52	2.56
	B_1	" "	1.45	1.81	1.89	2.20	1.88	2.45
	B_2	" "	1.79	2.09	2.33	1.75	2.12	1.60
	B_3	" "	2.04	2.08	1.66	1.56	1.60	1.22
	B_4	" "	1.88	1.94	2.08	2.28	1.88	1.95
9(*a*) duplicate of Test No. 9.	A_1	" "	2.47	2.78	1.81	2.01	1.76	1.61
	A_2	" "	2.42	2.78	1.56	2.11	1.27	1.28
	A_3	" "	2.05	2.40	1.30	1.69	1.14	1.16
	A_4	" "	2.32	2.41	2.47	2.14	2.55	2.56
	B_1	" "	1.70	2.08	1.90	2.13	1.86	2.48
	B_2	" "	2.14	2.18	2.46	1.63	1.99	1.58
	B_3	" "	2.23	2.00	1.95	1.64	1.43	1.18
	B_4	" "	1.95	1.93	1.94	2.28	1.75	2.01

Comparisons such as those presented in Table 1 demonstrate that meters may give vastly different registrations in turbulent water, but do not necessarily furnish the basis for an absolute comparison under like conditions, since the projected area of the propellers of the different types of meters was different in both size and shape. Although the utmost care may have been exercised to locate the center of the meter wheels in identical positions, and the regimen of the turbulent condition at any point may have been uniform, it cannot be assumed that the vanes themselves encountered exactly the same kind of stream filaments since the degree of turbulence varied throughout the cross-section of the channel. These conditions prompted further methods of investigations.

Since velocity measurements were taken at each measuring point (Fig. 10) with all six meters, a comparison was made of the mean of the velocities determined at the eight points by the various meters with the mean velocity computed from the weir discharge. These data are shown in Table 2. Although a greater number of observation points would have been used if actual discharge measurements had been made, the data show certain tendencies which were verified by later tests. It will be noted that the velocities as measured by the screw meters are usually less than their weir velocities, whereas those measured by the cup meters are generally greater.

EFFECT OF FLUCTUATIONS IN SPEED OF FLOWING WATER

The series of experiments described, served to show that a considerable degree of turbulence in the flowing water introduced large errors into the current-meter readings; but the results did not appear to be favorable for determining what modifications of type or design in the meter itself would give most promise of reducing these errors to a minimum. Furthermore, multiplying the number of such experiments did not seem to be a worthy procedure, largely because there is no existing method of measuring degree of turbulence and no standards or units in which turbulence may be expressed. Rather than leave the matter in this incomplete and unsatisfactory state, it appeared desirable to attempt to resolve the problem into simpler elements and to attack them separately.

TABLE 2.—PERCENTAGE VARIATION IN MEAN VELOCITY BY METER AS DETERMINED AT EIGHT POINTS FROM VELOCITY AS COMPUTED FROM WEIR DISCHARGE.

Test No.	Measuring conditions.	Mean velocity in flume as determined by weir.	PERCENTAGE VARIATION IN MEAN VELOCITY BY METER AS DETERMINED AT EIGHT POINTS, FROM WEIR VELOCITY.					
			Price.		Hoff 4-blade.	Ott.		
			Electric.	Acoustic.		Small.	Medium.	Large.
2	Vertical obstruction..	1.00	+54.2	+53.9	+27.0	+49.5	+43.7	+44.2
3	Horizontal paddle.....	1.00	+18.9	+26.8	−61.6	−17.4	−28.8	−20.9
4	Vertical "	1.00	−17.4	+18.5	−20.2	−15.8	−19.9	−14.9
5	Submerged weir.......	1.00	+23.3	+37.1	+27.5	+30.6	+25.0	+29.8
7	Vertical obstruction..	2.00	+ 4.8	+ 1.2	+ 4.2	+ 6.8	+ 3.8	−12.0
8	Horizontal paddle.....	2.00	+12.8	+30.4	−29.5	−12.6	−22.2	−15.4
8(*a*)	" "	2.00	+11.1	+16.7	−13.5	−12.9	−19.8	−12.1
9	Vertical paddle........	2.00	+ 5.1	+13.2	− 5.4	− 3.1	−13.5	−14.3
9(*a*)	" "	2.00	+ 9.5	+13.2	−11.2	− 1.3	−13.9	−18.9
10	Submerged weir.......	2.00	+38.4	+45.7	+30.1	+27.3	+29.3	+27.0

When a current meter is held in a turbulent stream, the action of the water against the rotating wheel may differ in two respects, and only two, from the action in straight stream-line flow: First, the velocity of successive portions of the stream coming into contact with the wheel may vary rapidly in amount; and, second, there may be similar rapid changes in the direction or obliquity with which the water encounters the wheel. This leads to the following two kinds of errors in the results: (1) Error in registration due to the failure of the meter propeller to respond accurately by rapid changes in speed to the rapidly changing momentum of the water as the eddy passes; and (2) error in registration due to the fact that the stream lines of the eddy, as it moves down stream, strike the propeller blade at rapidly varying angles. The latter is a condition far different from the parallel filaments, flowing normal to the meter section, which propel the rotor during its rating test in still water.

Referring to the errors in the first class, a heavy propeller would be slow in acceleration and possibly equally slow in retardation. It is considered desirable that a meter shall record the correct average velocity of the passing water irrespective of its instantaneous behavior under the varying impulses of the eddy. If the meter is held at each point a reasonably long period of time, it should record this correct average.

The second series of experiments was designed to evaluate the first of these two possible sources of error in registration. Each meter, mounted on a rod supported in a frame, was immersed at a definite point (Point x in Fig. 10), in a stream moving with a steady uniform straight-line velocity. The entire frame was then moved back and forth in a horizontal direction, parallel to the direction of flow at various speeds, by means of a crank and connecting rod operating through a stroke of 2 ft. Thus, instead of attempting to superimpose upon the stream of water definite and rapid changes in speed, an equivalent result was more easily accomplished by moving the meter itself, thereby producing, in the course of a test, many rapid fluctuations in the relative velocity of the water with respect to the meter. The determination of the accuracy of meters in registering the velocity of the filaments of an eddy which strike the meter at an angle, was reserved for the third and last part of this investigation.

The apparatus designed to create rapid changes in the relative velocity of the water with respect to the meter is shown in Fig. 9. The large wheel could be rotated at any desired speed, so as to cause the cross-head holding the meter rod to move back and forth through a stroke of 2 ft. in a definite manner approaching a simple harmonic motion. The connecting rod, however, was only 2 ft. long; hence, the motion of the piston was unsymmetrical and the piston attained a maximum velocity somewhat greater than that of the crank-pin. Although the operator attempted to rotate the wheel at a uniform speed, probably the down-stream stroke was completed in a slightly shorter period than the up-stream stroke, since greater resistance was encountered in moving the meter against the current, and there was very little fly-wheel effect in the mechanism.

All the meters illustrated in this paper except that in Fig. 1 were attached in turn to the apparatus and tested in four different quantities of flow, giving average velocities in the channel of 1, 2, 3, and 5 ft. per sec. Under each of these flow conditions, the meters were held 1 ft. below the surface of the water in the center of the channel, and the mechanism was rotated at speeds of 5, 10, 15, 20, 25, 30, 40, and 50 rev. per min.

Each period during which the propeller revolutions were counted lasted for at least 5 min. The times of intermediate contacts were also recorded, in order to make sure that the conditions were uniform and that the meter was functioning properly.

A complete set of observations on each meter under each condition of uniform velocity was made in five steps, as follows:

1.—The desired velocity in the flume was obtained by means of the weir.
2.—An observation was made with the meter held stationary.
3.—Observations were made at each of the rotating speeds mentioned, with occasional intermediate readings with the meter held stationary in order to check the uniformity of the local velocity at the fixed point in the channel.
4.—A final observation was made with the meter held stationary.
5.—Readings on the weir hook-gauge were taken before and after each observation to check the uniformity of the discharge through the channel.

The ratio of the average revolutions per second of the meter when oscillated to the corresponding speed of rotation when the meter is held stationary in the same current, is a measure of the degree to which the changes in relative water velocity affect the meter. The percentage of error in the registration of each of the meters thus obtained, plotted against the ratio of the crank-pin velocity to the uniform stream velocity, is shown in Fig. 12. The ordinates, "Relation of Meter Reading When Moving to Reading When Stationary", were computed from the number of revolutions, which approximately equals the ratio of stream velocities. The points plotted are the weighted averages for stream velocities of 2, 3, and 5 ft. per sec. When any points coincide, the symbols are moved to one side of the point.

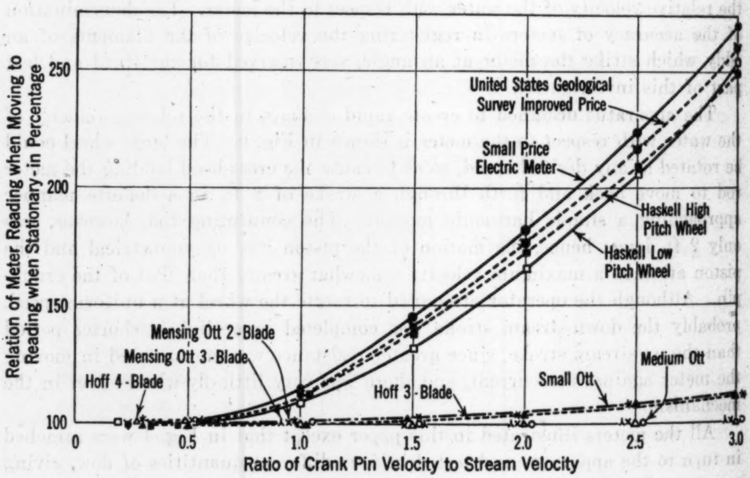


FIG. 12.—VARIATION IN RATIO OF READINGS WHEN METER IS MOVING TO READINGS WHEN METER IS STATIONARY. (WEIGHTED AVERAGE FOR STREAM VELOCITIES OF 2, 3, AND 5 FEET PER SECOND.)

Since the crank-pin velocity is almost equal to the maximum velocity of the cross-head operating the meter, the latter ratio is not only proportional to, but almost expresses, the ratio of the maximum internal velocity of the assumed eddy to the uniform velocity of the stream.

Fig. 12 indicates that the two Price meters and the two Haskell meters registered excessive velocities after the crank-pin velocity had approached the stream velocity in value, whereas the remaining screw meters showed little or no effect due to the oscillation of the meter. Below this point, where no reversal occurred in direction of the velocity of the water with respect to the meter, and thus the meters, at all times, were propelled by positive currents, the changes in relative water velocities produced less error in the registration of the meters. When the velocity of the meter did not exceed one-half the velocity of the water, they all behaved alike in integrating to obtain the direct average current velocity, regardless of the weight, shape, and size of the propeller. These tests show in reality that an eddy striking the meter "head on" does not affect the total counts of revolutions, provided the time of reading is not too short. According to the work of Hoff (cited in the Appendix), the weight of the propeller had an appreciable influence on the rating of the meter. Any practical difference did not become apparent in the Iowa tests.

When the maximum velocity of the backward stroke of the piston exceeded the stream velocity, a reverse current propelled the meters, during a part of the time. Under the impact of this current, the propellers of the Price meters continued to turn in a positive direction, thus over-registering; but the screw meters reversed their direction of rotation and, if no meter contacts were made while rotating in the backward direction, the reverse current was automatically subtracted from the excessive forward current measured during the up-stream stroke. With the proper contact mechanism there was practically no over-registration of this type of meter at any eddy speed.

Fig. 12 indicates some slight over-registration of the small Ott meter and the three-bladed Hoff meter when the maximum backward velocity exceeded twice the forward stream current. This was probably due in these two meters to the larger amount of interference to the reverse flow caused by the frames of the meters making the rotor move much more slowly in a reverse current than in a forward current of the same magnitude.

The two Haskell meters registered every single revolution of the propeller, which represented a travel of water past the meter of only 3.02 or 1.23 lin. ft., respectively. Moreover, the contact continued for more than one-half revolution; hence, in the back stroke of 2 ft., the reverse current made additional contacts which could not be distinguished from those made while rotating in the proper direction. The Haskell meters would have behaved the same as the other screw meters if they had been provided with registering devices which contacted every five revolutions or more. In fact, the other screw meters, which could be so adapted, behaved in a manner similar to the Haskell meter when operated on their single revolution contacts, registering a reverse current by contacts made in a distance less than the length of the back stroke of the piston. Tests under these conditions with the next to the smallest quantity of flow are shown in Fig. 13.

It is apparent, therefore, that the meters tested will not be influenced by successive variations in the speed of the flowing water as long as they are practically stationary and up-stream currents are not encountered. However, in case these backward currents do exist, the Price meters will invariably over-register and the screw meters will apparently do the same, unless some means of detecting the contact made in the reverse direction of rotation is adopted, or a contacting period is used, which is longer than the time in which the meter is under the influence of the backward current. If such a long interval between contacts is used, all the screw meters will perform with reasonable accuracy under ordinary changes in velocity.

EFFECT ON CURRENT-METER REGISTRATION OF STREAM FILAMENTS STRIKING THE PROPELLER AT VARIOUS ANGLES

The determination of the accuracy of meters in registering the velocity of the filaments of an eddy striking the meter at an angle was the object of the third part of this investigation. As in the previous experiments, the meters mounted on rods were held at the center line of the testing section. A pointer fastened to the rod and parallel to the axis of the meter enabled the operator to set the instrument so that the current in the testing canal struck the propeller at any desired horizontal angle (Fig. 9). In testing the Price meters with the current striking the propeller at various vertical angles, the meter rod was secured to the side of the cross-head of the machine as shown in Fig. 9. It could thus be turned to make the desired angle with the vertical.

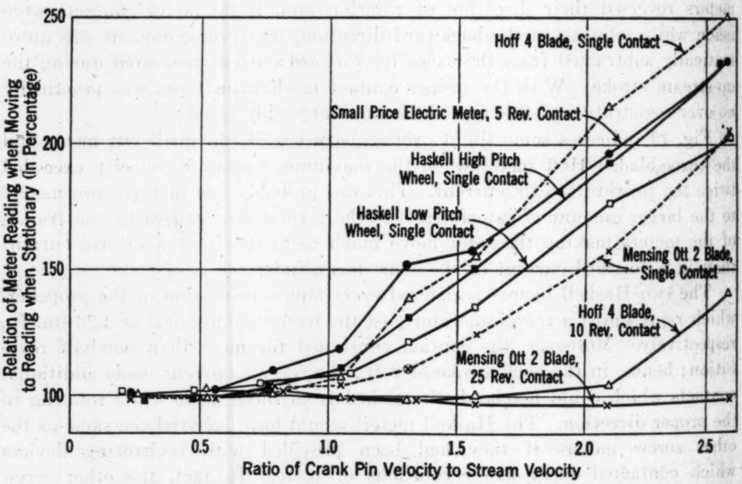


FIG. 13.—VARIATION IN RATIO OF READINGS WHEN METER IS MOVING TO READINGS WHEN METER IS STATIONARY, WITH A STREAM VELOCITY OF 2 FEET PER SECOND.

All the meters illustrated in Figs. 1 to 8, inclusive, were tested in turn in four different quantities of flow giving average velocities in the channel of 1, 2, 3, and 5 ft. per sec. Under each of these conditions of flow, the meters were held rigidly on a rod 1 ft. below the surface of the water, with the rod at the center of the channel. The velocity at the point at which each meter was held, was quite uniform as shown in Fig. 10. The tests were made first with the propeller of the meter turned to the right when looking up stream so that the meter axis made angles of 10°, 20°, 25°, 30°, 35°, 40°, and 45° with the normal position parallel to the filaments of flow.

The meters were then tested with their propellers turned to the left through the same series of angles with the maximum quantity of flow. These readings indicated that the performance of the three Price meters (Figs. 1, 2, and 8) and the medium Ott meter (Fig. 4(*b*)) departed farther from the ideal, with their propellers turned to the left than during the previous tests with their propellers turned to the right. Consequently, these four meters were tested through the entire series of angles when turned to the left in the same quantities of flow as those used in the tests with propellers turned to the right.

Apparently, the screw meters should perform practically the same in a current striking the propeller at a given angle regardless of whether the variation in angle is in a horizontal or a vertical direction, since a symmetrical face appears to be presented to the flow. This is not true of the cup meter. Hence, the three Price meters were tested in the same quantities of flow as those used in the horizontal angle tests, but with the meter axis tilted upward 5°, 10°, 15°, 20°, 25°, and 30° and also downward through the same series of angles.

Each observation during which the propeller revolutions were counted, lasted for at least 5 min. The times of all intermediate contacts were recorded in order to make certain that conditions were uniform and that the meter was functioning properly.

A complete set of observations on each meter under each condition of velocity included the following:

1.—A velocity reading was taken 1 ft. on each side of the center of the channel in order to check the uniformity of the horizontal velocity distribution.

2.—A reading was taken in the center of the channel with the meter in the normal position.

3.—Observations were made at each of the angles previously mentioned with occasional intermediate readings in the normal position to check the uniformity of the flow.

4.—A reading was finally taken with the meter in the normal position at the center of the channel.

5.—Readings of the weir hook-gauge were taken before and after each observation to check the uniformity of the discharge through the channel.

The tail-pieces of all the meters were removed when operated in the two maximum quantities of flow, because of the obstruction that they caused. The readings on a series of piezometers located near the bottom of one side of the channel were carefully observed when the large meters were turned at angles of 45° from the normal position in order to detect any appreciable change in channel depth caused by the obstruction of the meter. The maximum difference in depth under any condition did not exceed 0.002 ft.

The removal of the 4 by 4-ft. concrete flume at the conclusion of these tests afforded an opportunity to test the meters in a channel, 10 ft. wide, at a distance 120 ft. down stream from the head-gate. Tests duplicating those previously described in which the meters were displaced through horizontal angles, were made on several of the meters at various depths of submergence. No difference in behavior from that obtained in the smaller channel was noted.

When a current strikes the wheel of a meter in a diagonal direction, the ideal meter, if prevented from turning to face the oblique flow, should register in amount only the component of the velocity of the current which is parallel to the axis of the meter and normal to the gauging section. This component is equal to the actual velocity of the water multiplied by the cosine of the angle between the meter axis and the stream line of the current.

TABLE 3.—READINGS RECORDED BY ELEVEN DIFFERENT CURRENT METERS, HELD AT VARIOUS HORIZONTAL ANGLES WITH THE CURRENT. DEPARTURES OF THE RATIOS OF VELOCITY RECORDED BY METER AT VARIOUS ANGLES SHOWN AS A PERCENTAGE OF THE COSINE VALUE OF VELOCITY RECORDED BY METER, WITH AXIS PARALLEL TO THE CURRENT.

(Velocity of water, 5 ft. per sec.)

Type of meter.	Linear distance for one revolution of propeller, in feet.	Number of revolutions per contact.	Side of meter water approaches when looking up stream.	HORIZONTAL ANGLE, IN DEGREES.						
				10.	20.	25.	30.	35.	40.	45.
Hoff, 3-Bladed (rubber)	0.94	10	Right	+ 1.9	+ 0.3	— 2.7	— 7.4	—17.5	—24.6	—33.4
			Left	— 4.6	—15.2	—22.6	—33.6	—40.4	—44.7	—55.5
Hoff, 4-Bladed (rubber)	0.94	10	Right	— 0.7	— 5.7	—11.1	—16.7	—25.9	—37.2	—42.7
			Left	— 4.1	—14.2	—19.6	—26.9	—38.2	—47.0	—54.4
Price, No. 211 880	2.23	5	Right	+ 1.7	+ 5.2	+11.2	+15.3	+23.1	+31.6	+44.5
			Left	+ 1.0	+ 3.4	+ 6.2	+10.4	+15.9	+21.6	+29.3
Price, Acoustic	2.24	10	Right	+ 0.7	+ 5.5	+ 9.8	+14.6	+21.0	+29.0	+40.7
			Left	+ 0.7	+ 3.9	+ 6.1	+10.3	+15.5	+21.5	+28.9
U. S. Geological Survey Improved Price	2.24	5	Right	— 0.2	+ 3.9	+ 6.0	+12.0	+19.2	+30.6	+41.4
			Left	+ 2.1	+ 6.2	+ 7.5	+11.4	+16.7	+20.2	+31.0
Small Ott	0.41	50	Right	+ 1.8	+ 5.3	+ 5.0	+ 3.2	+ 1.2	— 4.2	— 7.3
			Left	+ 1.2	+ 0.8	— 2.8	— 5.4	—11.1	—13.2	—15.4
Medium Ott	0.85	25	Right	— 2.0	— 7.4	—11.1	—17.3	—26.1	—36.4	—41.7
			Left	— 0.4	— 4.8	— 8.6	—13.7	—20.4	—26.8	—34.8
Mensing-Ott, 2-Bladed	1.71	25	Right	— 1.7	— 6.9	—10.9	—11.7	—19.9	—20.7	—23.8
			Left	— 5.3	—10.9	—17.3	—23.7	—29.5	—33.0	—33.0
Mensing-Ott, 3-Bladed	1.72	25	Right	+ 0.2	— 2.0	— 3.5	— 6.2	—10.4	—13.5	—18.1
			Left	— 2.6	— 7.7	— 9.1	—11.5	—19.3	—24.2	—32.6
Haskell, Type B, high-pitch wheel	3.02	1	Right	— 3.5	—14.9	- 24.8	—33.5	—43.3	—46.5	—39.5
			Left	— 4.0	—15.9	—24.2	—36.4	—45.0	—47.1	—36.4
Haskell, Type B, low-pitch wheel	1.23	1	Right	— 1.6	— 6.6	— 8.0	—16.8	—24.5	—28.4	—31.4
			Left	— 2.9	— 9.6	—16.4	—20.6	—27.9	—32.2	—35.4

In making the tests with the water striking the propeller at various horizontal angles, most of the meters showed a slightly different performance when the water approached the meter from the left than when it approached at the same angle from the right. The results of the tests made on all the meters using the maximum velocity of flow (the water approaching the meter from the left and then from the right) are shown in Table 3. The Price meters over-registered every current that approached from any appreciable angle in a horizontal direction. The performance of the screw meters varied somewhat, all of them under-registering currents approaching at large angles.

When tested in the four different velocities of flow, the same meter generally showed a slight variation in performance for the same angles. The extent of the difference is indicated in Fig. 14 for one of the cup and one of the screw meters. While the variation is not large, repeated checks demonstrated that the difference observed was not accidental, but that there was a significant variation in the performance of the meters under the influence of currents of different velocities.

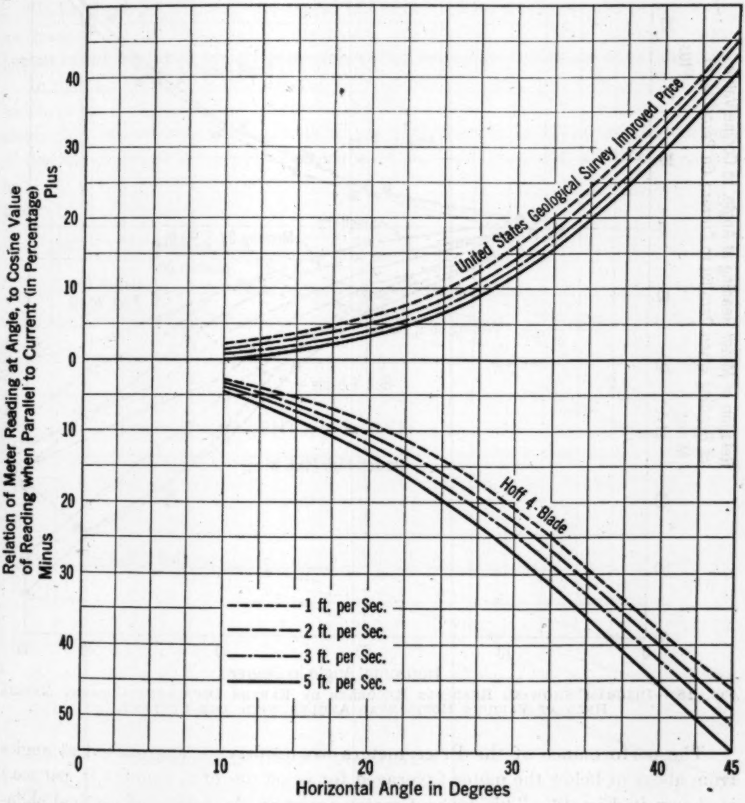


FIG. 14.—DIAGRAM SHOWING EFFECT OF HOLDING TWO TYPES OF METERS AT VARIOUS HORIZONTAL ANGLES WITH THE DIRECTION OF THE CURRENT.

A weighted average of the results secured from each meter when using velocities of 2, 3, and 5 ft. per sec., was computed by giving each separate result a weight proportional to the velocity under which it was obtained, and taking the water approaching the meter from either side gave the greater divergence from the ideal. The maximum erroneous effect of oblique currents thus revealed is plotted in Fig. 15.

At large current angles, the Price meters over-registered, and all screw types registered less than the forward component of velocity.

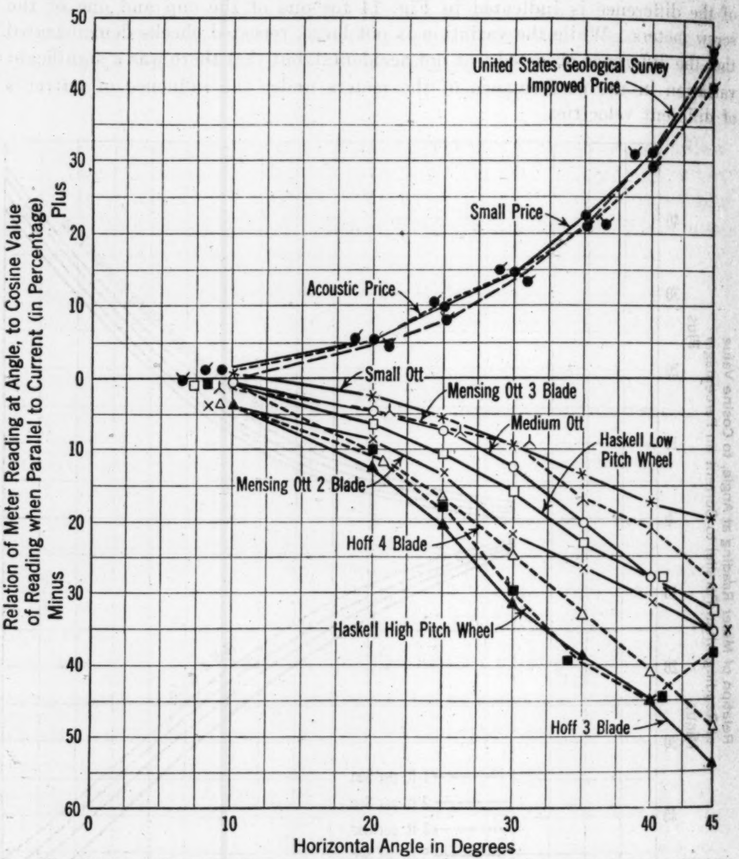


FIG. 15.—DIAGRAM SHOWING READINGS RECORDED BY ELEVEN DIFFERENT CURRENT METERS HELD AT VARIOUS HORIZONTAL ANGLES WITH THE CURRENT.

The performance of the Price meters when currents approached at angles from above or below the meter (averaged for velocities of 2, 3, and 5 ft. per sec.) is shown in Fig. 16. This type of meter registers the cosine of vertical obliquities with much better accuracy than the horizontal, showing a slight tendency toward under-registration of these currents. The degree of under-registration of the component vertical to the gauging section in vertical currents, however, is not nearly as great as the amount which this meter over-registers these components in measuring horizontal disturbances.

No important difference was observed in the performance of the three types of Price meters in these tests, and probably no other type of meter will register more accurately under conditions of vertical obliquities.

In a turbulent stream, where filaments strike the meter with equal frequency from all angles, the meters would perform on the average as shown in Fig. 17. Where such a condition exists, it should be remembered that vertical and horizontal angles develop compensating effects in the cup meter. The curves shown are obtained by averaging the performance of the meter in the four directions measured in these tests. Under this condition of turbulence, the Price meter compares more favorably with the others than it did in Fig. 15.

Under these conditions, the small Ott meter (Fig. 4(*a*)) would prove to be an instrument of remarkable accuracy, registering the true average component of all currents which approached the meter at angles up to 20 degrees.

In making measurements around débris, bridge piers, and side contractions which cause horizontal disturbances, the Price meter will over-register appreciably; but in metering in channels where disturbances originate at the bottom of the stream, causing a preponderance of vertical currents, the Price meter held on a rod should register the desired velocity with a good degree of accuracy.

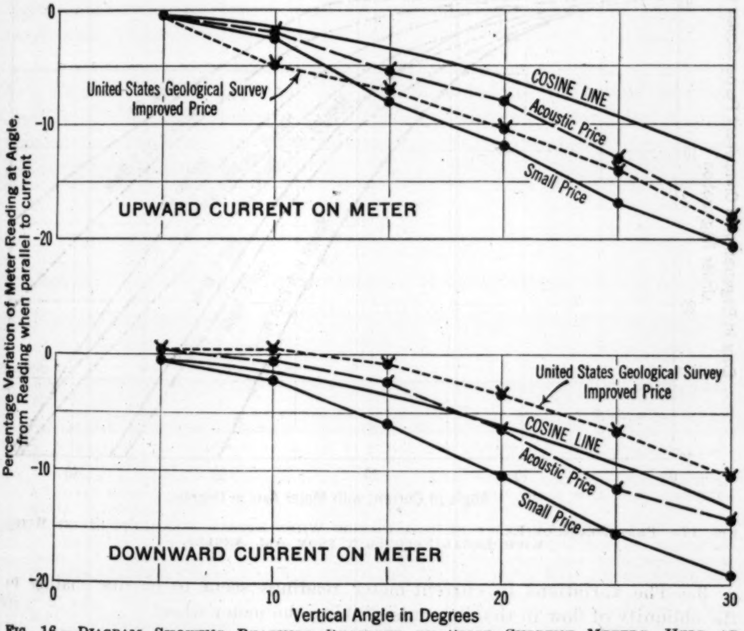


FIG. 16.—DIAGRAM SHOWING READINGS RECORDED BY PRICE CURRENT METERS, HELD AT VARIOUS VERTICAL ANGLES WITH THE CURRENT. WEIGHTED AVERAGE FOR STREAM VELOCITIES OF 2, 3, AND 5 FEET PER SECOND.

CONCLUSIONS AND LIMITATIONS

The following general conclusions seem to be justified:

1.—A flowing stream may be so turbulent as to change very noticeably the readings obtained by a current meter.

2.—The disturbances in the current-meter readings are not caused to any noticeable degree by the fact that the flow is unsteady; that is, they are not influenced by the inertia of the current-meter wheel, nor are they produced necessarily when there are pulsations or any unsteadiness in the stream.

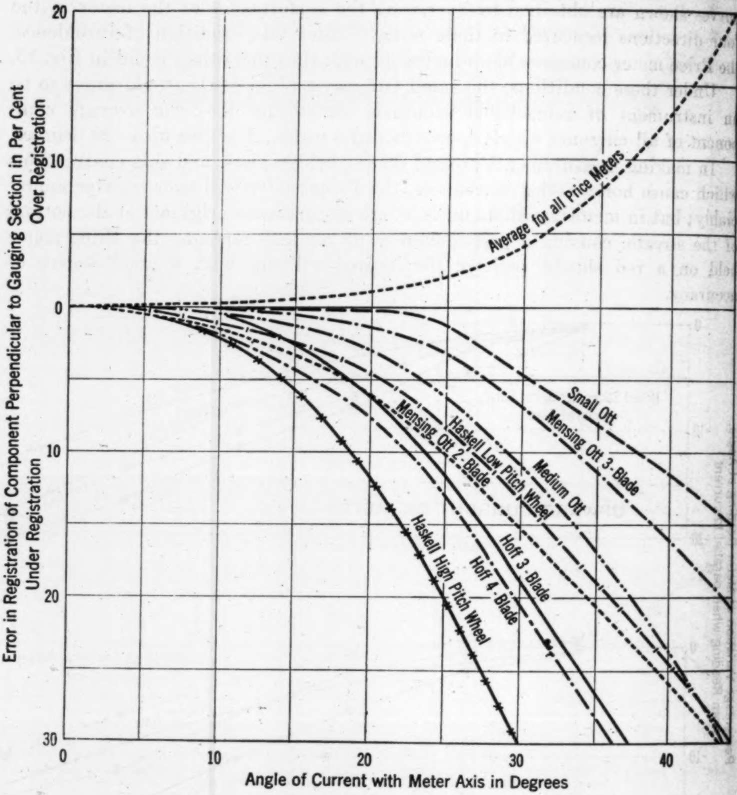


FIG. 17.—PERCENTAGE OF ERROR IN REGISTRATION WHEN STREAM FILAMENTS STRIKE METER WITH EQUAL FREQUENCY FROM ALL ANGLES.

3.—The variations in current-meter readings seem to be due mainly to the obliquity of flow in the filaments striking the meter wheel.

4.—When meters are held on a rod in a rigid position, with the meter axis parallel to the general axis of the stream, turbulent flow invariably causes the cup meter to over-register and the screw type to under-register.

5.—The cup meter registers the true velocity of currents approaching the meter from almost every horizontal direction; hence it over-registers the component perpendicular to the gauging section of horizontally oblique flow. Obliquity of flow in a vertical direction has a slight tendency to cause under-registration and thus may compensate to some degree for the over-registration caused by horizontal obliquity.

Certain limitations should be mentioned. This study was not an attempt to make a careful or sweeping comparison of meters produced by different manufacturers, and the figures shown should not be used for this purpose.

It should be remembered also that rod support was used in all these experiments, and the conclusions must not be applied without modification to cable-suspended meters. In a majority of cases, stream gauging is done with cable-suspended meters. In such suspension the tail-vanes tend to keep the meter squarely facing the current regardless of any local or temporary obliquity of flow. In some places, however, such as close to the sharp corners of piers or obstructions, the curvature of the filaments may be so sharp that the direction of flow at the meter wheel is appreciably different from that at the vane. In such case oblique flow must necessarily strike the current wheel. Furthermore, the dynamic center of some meters does not coincide with the mechanical center. Since the suspension by cable comes in a line with the mechanical center, there may be sufficient eccentricity to cause the meter to swirl unduly in the water.

Finally, it is to be remembered that the question of accuracy of registration, discussed in this paper, is only one of numerous important practical questions arising in the construction and use of current meters. Other questions of importance are cost, convenience, portability, durability, consistency, uniformity, reliability, etc.

APPENDIX

BIBLIOGRAPHY OF PUBLISHED REFERENCES ON CURRENT-METER CHARACTERISTICS BY AMERICAN INVESTIGATORS

Humphreys, A. A., and Abbot, H. L. Report on the Physics and Hydraulics of the Mississippi River. *Professional Paper No. 4,* Corps of Topographical Engineers, U. S. Army, 1861, pp. 203, 224.

Mentions current meter as a possible device for use on study of Mississippi River, but rejected same with preference expressed for floats.

Henry, D. Farrand. On the Flow of Water in Rivers and Canals. *Journal,* Franklin Inst., Vol. XCII, 1871, pp. 167-173, 257-262, 322-328, 383-389.

Describes various water-measuring devices, including his electrical attachment to a current meter. Gives results of meter ratings and comparisons with float measurements. Discusses many natural phenomena of stream flow.

Ellis, T. G. Report, Chief of Engineers, U. S. Army, 1878, Pt. I, pp. 305-350.

Describes measurements made on discharge of Connecticut River by means of floats and current meters. A screw type of meter (Woltman) was used and also a cup type of meter (Ellis) was first constructed for this work, both giving concordant results with the float measurements.

Blum, Richard. The Current Meter of Professor A. R. Harlacher. *Engineering News,* Vol. IX, 1882, pp. 141-142.

Describes the Harlacher screw type of electric current meter.

Fteley, A., and Stearns, F. P. Description of Some Experiments on the Flow of Water Made During the Construction of Works Conveying the Water of Sudbury River to Boston. *Transactions,* Am. Soc. C. E., Vol. XII (1883), pp. 117-118.

Describes current meter used in making comparative measurements with weir, and states the agreement was within 1% unless the velocity was very irregular. Extreme agitation of the current meter during rating caused an error of 9 per cent.

Stearns, F. P. On the Current-Meter, Together with a Reason Why the Maximum Velocity of Water Flowing in Open Channels Is Below the Surface. *Transactions,* Am. Soc. C. E., Vol. XII (1883), pp. 301-338.

Illustrates and describes the Fteley and Stearns screw type of current meter. Gives results of rating experiments in which an irregular and variable velocity was created by oscillating the meter; and experiments during which the axis of the meter was turned at various angles with the direction of motion of the rating car up to a maximum of 41°, in which position the meters under-registered about 10 per cent. Results of twenty-seven experiments are given in which the discharge of the Sudbury Conduit was measured both by current meter and weir with varying degrees of agreement depending mostly on the method used in taking the velocity observations by current meter.

The Price Current Meter. *Engineering News*, Vol. XXIX, 1893, pp. 196-197.

Describes large Price meter invented by the late W. G. Price, M. Am. Soc. C. E., and patented August 25, 1885. Follows article describing current-meter measurements on Niagara River.

Price, W. G. A New Current Meter and a New Method of Rating Current Meters. *Engineering News*, Vol. XXXIII, 1895, p. 27.

Describes Price acoustic current meter and method of rating from skiff in still pond.

The Ritchie-Haskell Direction-Current Meter. *Engineering News*, Vol. XXXIII, 1895, pp. 27-28.

Describes the Haskell current meter and also direction-recording apparatus by which the direction of the current may be observed.

Noble, Theron A. Gauging of Cedar River, Washington. *Transactions*, Am. Soc. C. E., Vol. XLI (1899), pp. 1-26.

Describes the use of a Haskell meter in measuring stream flow, and compares same with measurements by floats. The discussion as to the accuracy of the measurements has historical interest.

U. S. Geological Survey. Annual Report, 21st, 1899-1900, Pt. 4, Hydrography, pp. 28-29.

Describes and illustrates by two diagrams a screw type of current meter known as the Moulinet, Type H.

Haskell, E. E., Shenehon, F. C., and Sabin, L. C. Report of the Chief of Engineers, U. S. Army, 1900, pp. 5322-5401.

Describes measurements of Niagara and St. Clair Rivers in which Haskell meters were rated and used, with various comments as to their reliability.

White, W. M. The Pitot Tube, Its Formula. *Journal*, Assoc. of Eng. Societies, Vol. 27, 1901, pp. 64-66.

Uses Price current meter to compare with Pitot tube in rating experiments by towing through still water, showing excellent agreement between the two instruments.

Horton, Theodore. Flow in the Sewers of the North Metropolitan Sewerage System of Massachusetts. *Transactions*, Am. Soc. C. E., Vol. XLVI (1901), pp. 78-92.

Describes the use of an Ellis (cup type) meter in measuring the discharge in a sewer. Mr. Horton's closing discussion mentions the possibility of erroneous registration where stream filaments of different velocities act on the two sides of the propeller.

Murphy, E. C. Tests to Determine the Accuracy of Discharge Measurements of New York State Canals and Feeders. *Water Supply and Irrigation Paper No. 47*, 1901, U. S. Geological Survey, pp. 18-29.

Describes comparative measurements by rods and meter on New York State canals and also experiments made in 1900 with rods, small Price meters, and weir at Cornell University Hydraulics Laboratory.

U. S. Geological Survey. Methods of Stream Measurement. *Water Supply and Irrigation Paper No. 56*, 1901.

Describes various methods of stream measurement, including use of the current meter. The Price meter is described in detail.

Murphy, E. C. Accuracy of Stream Measurements. *Water Supply and Irrigation Paper No. 64*, 1902, U. S. Geological Survey, pp. 11-95.

Discusses difficulties, and instruments used in measuring stream velocities, giving history of current meter, describing various types, and stating their advantages and disadvantages. Discusses the effect of pulsation of velocities of moving water on measurements. Extracts results of Ellis, Marr, Henry, Gordon, New York State Canal Survey, Fteley and Stearns, comparing float measurements with weir. Describes in detail his own experiments at Cornell University and Chevy Chase, Md., in 1900-1901. Compares measurements with Price, Haskell, and Fteley meters with weir measurements. Concludes that small Price current meters under ideal conditions can measure discharge within 1 or 2%; is more accurate when held rigidly on a rod rotating faster under this condition; will under-register current velocities closer to the water surface than 0.5 ft., etc. Some experiments under abnormal conditions showed results departing as much as 40% from the weir discharge. Haskell meter registered velocities near the surface more accurately than Price meter, not as accurate in low velocities.

Williams, G. S., Hubbell, C. W., and Fenkell, G. H. Experiments on Flow of Water in Pipes. *Transactions*, Am. Soc. C. E., Vol. XLVII (1902), pp. 337-341.

Occasional reference to meter ratings in flowing water as compared with still water, with particular reference to its accuracy as compared with Pitot tube. In discussion, E. C. Murphy (pp. 202-203) gives results of eighteen measurements with Price meters in moving water at Cornell University showing agreement with standard weir within 2.8% in every case; D. Farrand Henry (pp. 224-226) describes early experiments comparing velocities obtained by floats with that registered by meters; George Y. Wisner, (pp. 231-233) discusses error in registration of meters in flowing water; E. E. Haskell (pp. 283-289) describes rating tests in flowing water which gave favorable comparison between a Haskell meter and floats; and Robert E. Horton (pp. 289-291) discusses nature of eddies and the performance of current meters.

Murphy, E. C. Current Meter and Weir Discharge Comparisons. *Transactions,* Am. Soc. C. E., Vol. XLVII (1902), pp. 370-391.

Describes fifty discharge measurements with Haskell and Price meters in testing canal at Cornell University with agreement between meters closer than with weir, but in every case variations were less than 4.8% ; twenty of the tests showed differences less than 1 per cent. In discussions: Charles H. Miller (pp. 379-380) discusses comparative results with different current meters, describing experiments with Haskell and Price meters mounted on a skiff which was rocked, producing over-registration of the Price meter and under-registration of the Haskell meter; J. B. Lippincott (pp. 383-387) describes eleven experiments comparing Price and Haskell measurements, also, in some cases, with weir; mentions possible error involved in integration method using Price meter; and E. E. Haskell (pp. 387-388) expresses his opinion of the relative merits of screw type and cup type of meters.

Murphy, E. C., Hoyt, J. C., and Hollister, G. B. Hydrographic Manual of the United States Geology Survey. *Water Supply and Irrigation Paper No. 94,* 1904, U. S. Geological Survey, pp. 19-31.

Discusses use of meters in obtaining discharge measurements, and describes Price meters in detail.

Murphy, E. C. Accuracy of Stream Measurements. *Water Supply and Irrigation Paper No. 95,* 1904, U. S. Geological Survey, pp. 1-169.

Revision of *Water Supply Paper No. 64* with additional experimental data comparing Price and Fteley meters in shallow streams of varying roughness with weir measurements. The part giving data on vertical velocity curves has been augmented by additional data.

Barrows, H. K. Work of Hydrographic Branch, United States Geological Survey, in New England, and Discussion of Methods for Estimating Stream Flow. *Journal,* Assoc. of Eng. Societies, July, 1905, pp. 19-20.

Describes work of the U. S. Geological Survey on surface water resources in New England, including current practice in securing discharge measurements with Price meter.

Senate Document 105, 62d Congress, 1st Session, pp. 72-74.

Describes an experiment by F. C. Shenehon, M. Am. Soc. C. E., with a Haskell meter in the Detroit River in 1906, comparing the velocity registered by this meter with that observed from the rate of travel of liquid blueing. Good experiment was obtained.

Gregory, W. B. Test of a Rotary Pump. *Transactions,* Am. Soc. Mech. Engrs., Vol. 28 (1907), pp. 745-769.

Gives results of pump discharges measured in a flume by both Price electric current meter and a Tulane Pitot tube. Individual measurements showed a maximum variation of 2% between the results obtained in measuring about 153 cu. ft. per sec. with the two different instruments and an average variation of but 0.1 per cent.

Hoyt, John C. A Universal Current Meter. *Engineering News,* Vol. 57, 1907, p. 263.

Describes adaptability of small Price meter to stream gauging, and gives rating curves for Fteley, Warren, Haskell, small Price, large Price, and Lallie current meters.

Stevens, J. C. Comparison of Formulas for Computation of Stream Discharge. *Engineering News,* Vol. 59, 1908, pp. 682-684.

Discusses errors involved in various methods of using current-meter data in obtaining the discharge of a stream. Recommends the method adopted by the U. S. Geological Survey, and also multiplying each depth by its corresponding velocity.

Hoyt, John C. Recent Changes of Methods and Equipment in the Water Resources Work of United States Geological Survey. *Engineering News,* Vol. 60, 1908, pp. 15-16.

Describes refinements which were made in construction and use of Price meter, referring to pentacount registering device, torpedo weights, rods, etc.

Hoyt, John C. The Use and Care of the Current Meter as Practiced by the United States Geological Survey. *Transactions,* Am. Soc. C. E., Vol. LXVI (1910), pp. 70-105.

Gives history of current-meter development, plates showing Price, Haskell, Fteley, Ellis, Richards, Woltman, Ott, and Amster meters. The use, care, and rating of the Price meter is thoroughly discussed. In discussion, A. H. Dramont (pp. 106-107) describes gaugings and ratings of Fteley meter in measuring the discharge of Croton Aqueduct; E. C. Murphy (pp. 107-108) mentions the possibility of the different behavior of a meter near the sides and bottom of a channel; H. K. Barrows (pp. 108-110) stresses certain precautions in use of the Price meter; C. E. Grunsky (pp. 110-126) discusses the difference in results obtained in computing discharges from average of velocity observations taken at the end of sections as practiced by the U. S. Geological Survey in contrast to the method proposed by W. H. Hall, M. Am. Soc. C. E., in 1878 of multiplying velocity measurements by depth at the point of the measurement. Mentions possibility of error in meters of various types and other gauging problems; E. Kuichling (pp. 126-128) suggests various possibilities of error in the rating and registration of meters; G. H. Matthes (pp. 128-129) concludes that the precision of the current meter is far ahead of other observations made in obtaining discharge data on natural streams; Walter Pearl (p. 130) agrees with the conclusion of Mr. Matthes; and C. M. Allen (pp. 130-132) expresses confidence in measurements with the current meter, based on experiments at the Hydraulic Laboratory of the Worcester Polytechnic Institute and elsewhere.

Van Winkle, Franklin. Stream Flow at Single Cross-Section. *Power,* Vol. XXXII, Pt. 2, 1910, pp. 1553-1557.

Describes pressure plate, Pitot tube, float wheel, and current meters. Gives detailed discussion of Price and Ott meters. Use and advantages of each.

Stevens, J. C. Hydrometry as an Aid to the Successful Operation of an Irrigation System. *Transactions,* Am. Soc. C. E., Vol. LXXI (1911), p. 323.

Gives results of twelve measurements made for the purpose of comparing the small Price pentacount head with the single count head. The maximum difference was 1.8 per cent.

Williams, Gardner S. Measurement of Water. *Journal,* Western Soc. of Engrs., Vol. XVII, 1912, pp. 22-48.

Discusses various devices, including current meters, mentioning comparison with float measurements at Cornell University and work of U. S. Lake Survey.

Hosea, R. M. Measurement of the Flow of Water in Large Pipe Lines. *Journal,* Western Soc. of Engrs., Vol. XVII, 1912, pp. 34-39.

Describes special Price meter of electrical and mechanical type used on rods inside 28-in. wood-stave pipe. Rating curves for same are shown.

Groat, B. F. Characteristics of Cup and Screw Current Meters. *Transactions,* Am. Soc. C. E., Vol. LXXVI (1913), pp. 819-840, 852-870.

Describes tests made with Haskell and Price current meters and Pitot tube at Massena, N. Y., concluding that cup meters over-register (on the average 6%) more than screw meters under-register (the average for Haskell was about 1 per cent.). States that current–meter measurements in perturbed water should be made with caution. Results of experiments given in which meters were oscillated longitudinally and transversely through a small amplitude in a still-water rating, and also some ratings from a skiff with oscillations caused by rocking the boat. An addendum states that experience on some mountain streams indicates that cup meters will over-register three to six times the amount which the screw meter under-registers. In discussion, W. G. Price (pp. 841-844) defends the Price meter and states its merits; E. E. Haskell (pp. 844-846) states merits of screw meter and gives eight comparative measurements between the velocities obtained by meters and floats; C. H. Miller (pp. 846-848) stresses accuracy in rating meters; and J. C. Hoyt (pp. 848-852) gives reasons for the U. S. Geological Survey adopting the Price meter and concludes that it will give reliable results at all but the most exceptional stations.

Rumpf, Charles P. An Investigation of the Use and Rating of the Current Meter. *Engineering News,* Vol. 71, 1914, pp. 1083-1084.

Gives results of tests on cup and screw meters holding meters at various angles with the current.

Groat, B. F. Pitot Tube Formulas, Facts and Fallacies. *Proceedings,* Engrs.' Soc. of Western Pennsylvania, Vol. 30, 1914-15, pp. 323-383.

Paper devoted mostly to Pitot tube, but on pages 351-366 gives results of ratings of large Price and Haskell current meters from boats under ideal conditions and with boat rocking; describes use of these meters in turbine tests at Massena, N. Y. Concludes that cup meter was accelerated by rocking and screw meter retarded.

Moody, Lewis F. The Measurement of the Velocity of Flowing Water. *Proceedings,* Engrs.' Soc. of Western Pennsylvania, Vol. 30, 1914-15, pp. 280-323.

Paper devoted mostly to Pitot tube, but on pages 320-322, the results of comparative ratings of a Pitot tube, Price meter, and Fteley and Stearns meter are compared with the meters turned through various horizontal angles up to 70 degrees. Also gives results of an experimental screw meter with good characteristics. Concludes cup meter over-registers and registers velocities from right differently from those from left; screw meter under-registers. In discussion, Gardner S. Williams (p. 386) describes results of rating of Haskell meter by weir in turbulent water in which Haskell under-registered more than 3%; N. C. Grover (pp. 391-397) discusses work of the U. S. Geological Survey and adaptability of Price meter to this work; efforts to develop screw meter; and general history of Price meter; Thomas P. Roberts (pp. 397-400) remarks generally on some difficulties encountered with current meters; Charles M. Allen (pp. 401-402) describes Worcester Polytechnic Institute circular rating station; Clemens Herschel (p. 410) remarks generally to the effect that current-meter gauging is only adapted to the measurement of large open channels; E. H. Brown and F. Nagler (pp. 415-424) give account of tests made on large and small Price current meters operated at center of 42-in. main, rotated at various angles with the thread of the current, through 360° in horizontal plane and 180° in vertical plane. Curves for small Price meter show different registration to left from that of right, over-registration at all angles in horizontal plane, and satisfactory or under-registration for angles in vertical plane. Curves for large Price meter show its inferiority to small Price meter; Lewis F. Moody (pp. 454-455) gives results from Rensselaer Polytechnic Institute thesis on Price meter inclined at various horizontal and vertical angles up to 12°, confirming Brown and Nagler tests; and B. F. Groat (pp. 460-473) further discusses the Massena current-meter work, design of ideal meter, and mentions same having been produced at Rensselaer Polytechnic Institute. Comments on Haskell meter, and effect of bearing friction in current meters, possible inertia effect of heavy propellers.

Scobey, Fred C. Behavior of Cup Meters Under Conditions not Covered by Standard Ratings. *Journal of Agricultural Research,* May 25, 1914, pp. 77-83.

Gives results of experiments on Price current meter rated in still water on rod and cable, tipped varying amounts to 30° upward and downward, also near water surface, bottom, and channel sides, with dull pivot, and vertical oscillation of meter. Concludes meter should be held horizontal, channel sides have little effect, but bottom and surface readings are in error; dull pivot has inappreciable effect for velocities greater than 1 ft. per sec.; vertical meter movement rotates the meter in the same direction as moving water.

Harding, S. T. Experiments in the Use of Current Meters in Irrigation Canals. *Journal of Agricultural Research,* November 8, 1915, pp. 217-232.
Comparisons of the relative accuracy of discharge measurements made in irrigation canals with current meters using different methods of measuring velocities in the verticals are discussed. No type of meter is mentioned.

Groat, B. F. Chemi-Hydrometry and Its Application to the Precise Testing of Hydro-Electric Generators. *Transactions,* Am. Soc. C. E., Vol. LXXX (1916), pp. 1231-1271.
Describes still-water rating experiments on Haskell, Ott, and Price meters at University of Michigan, in which the meters were oscillated at various periods through arcs of various lengths in a horizontal, vertical, and longitudinal direction. Longitudinal oscillations showed that water pulsations affect meters but little, while lateral disturbances made the Price meter over-register, whereas the Ott and Haskell meters under-registered, the under-registration of the Haskell meters exceeding that of the Ott type. In discussion, Robert E. Horton (pp. 1283-1285) discusses accuracy of current-meter measurements; and W. S. Richmond (p. 1287) mentions several comparative measurements between current meters and other methods, and instances of erroneous registration of both types of meters.

Hoyt, J. C., and Grover, N. C. River Discharge. Fourth Edition, 1916, pp. 6-22.
Discusses history of and use and care of current meter with particular reference to the Price meter, duplicating article by J. C. Hoyt, *Transactions,* Am. Soc. C. E., Vol. LXVI (1910), p. 70.

Blanchard, Murray. Hydraulics of the Chicago Sanitary District's Main Channel. *Journal,* Western Soc. of Engrs., Vol. XXV, 1920, pp. 483-506.
Describes measurements with Haskell and Price meters under a variety of field conditions with data from occasional still-water ratings made during the progress of the work. Concludes that the Haskell meters appeared more accurate and reliable than the cup meters.

Fortier, S., and Hoff, E. J. Defects in Current Meters and a New Design. *Engineering News-Record,* Vol. 85, 1920, pp. 923-924.
Discusses the inaccuracies of meters under field conditions and describes the new Hoff meter giving curves comparing the effect of friction in various types of meters.

Gutmann, O. Glorifying the Current Meter. *Engineering News-Record,* Vol. 97, 1926, p. 840.
Review of "Der Genauigkeitsgrad von Flügelmessungen bei Wasserkraftanlagen", by Dr. Ing. A. Staus, in which the propeller type of meter is shown to be more reliable than the cup type.

Hoyt, K. K. Defense of the Price Current Meter. *Engineering News-Record,* Vol. 97, 1926, p. 1010.
Letter defends the use of Price meter by the U. S. Geological Survey, comparing the behavior of same with screw meters advocated by Dr. Ing. A. Staus.

Matthes, G. H. Price Current Meter: Serviceability *vs.* Scientific Design. *Engineering News-Record,* Vol. 98, 1927, p. 126.
Letter stresses the fact that the hydrographers of the U. S. Geological Survey during the early existence of the Bureau found the Price meter to be more serviceable than other types.

Groat, B. F. In Defense of Current Meters. *Engineering News-Record,* Vol. 98, 1927, p. 370.
Letter defends current meter as a valuable measuring device, and mentions characteristics of different types.

Liddell, William A. Stream Gauging. First Edition, 1927, p. 9 and pp. 136-150.
Discusses experimental work done in the United States, the effects of turbulent flow, and the behavior of cup-shaped and screw-shaped meters when subjected to various tests.

DISCUSSION

CHARLES S. BENNETT,[4] M. AM. SOC. C. E. (by letter).—The data presented in this paper are of particular interest to hydrographers because they make possible a quantitative comparison of the effects of turbulence in streams on the various types of meters. For many years there has been considerable discussion among hydraulic engineers regarding the relative merits of the different types of meters, and the data obtained in these experiments should give conclusive evidence with respect to some of the various contentions. While a few similar experiments have been made heretofore, it appears that the work done in this series more nearly approaches actual field conditions than any previous attempts.

It has long been recognized that meters of the cup type will over-register when exposed to transverse or reverse currents and that the screw-type meters tend to under-register under similar conditions. It is interesting to note that the percentage of over-registration of the Price or cup type of meters for various horizontal angles does not greatly exceed that of under-registration in the screw-type of instruments. It would appear that, disregarding all other considerations, all types of meters now in general use (with the exception of the Haskell, high-pitch wheel, and the Hoff) are subject to about the same degree of inaccuracy under these conditions, to horizontal angles of about 30 degrees. However, meters of the cup type tend to under-register under the influence of obliquity of flow in a vertical direction. Since turbulence causes obliquity vertically as well as horizontally, it follows that the cup meter will probably show less actual over-registration in a turbulent stream than is indicated by the data in Fig. 15.

Stream-flow measurements with current meters are made ordinarily from cables or bridges at selected points, and the proportion of the total cross-section in which turbulence occurs due to piers or obstructions is relatively small. When measurements are carefully made at such sections, the results should be reasonably accurate, no matter what type of standard meter is used. However, there are so many practical advantages in the improved Price meter used by the U. S. Geological Survey, that it seems to be the most satisfactory for general purposes.

It might be suggested that, where it is necessary to make measurements in races or other streams in a turbulent condition, readings be obtained with a cup meter as well as with a screw meter, such as the small Ott. Comparisons of the readings should give an idea of the approximate compensation to allow for inaccuracy due to angular approach of the stream filaments. It might also be well to make longer observations than usual when turbulent conditions exist; that is, make each reading of velocity cover a period of about 180 sec. instead of the customary 60 to 70 sec.

The data in Tables 1 and 2 result from experiments made at velocities of 2 ft. per sec., and less. From Fig. 14 it appears that an increase of velocity up to 5 ft. per sec. makes a considerable difference in the percentage of variation for two types of meters at various horizontal angles. This comparison raises the question of the effect of higher velocities, such as occur in ordinary streams in flood, on the variation. It is hoped that further studies may be made, using experiments at velocities somewhat higher than those of this series.

---

[4] Engr., The Miami Conservancy Dist., Dayton, Ohio.

The writer believes that these experiments bear out the general opinion of hydrographers that current meters cannot be expected to register correctly in turbulent water; but it should be remembered that the greatest practical value of the current meter lies in its use for the measurement of the flow of average streams, in which it is usually possible to select a measuring section where there is a minimum of disturbing conditions.

Carl Rohwer,[5] Assoc. M. Am. Soc. C. E. (by letter).—The current meter is frequently the only available means of measuring the discharge of natural streams or artificial channels, and any information that bears on this subject is always welcomed by those who are concerned with the measurement of water. The authors have carried on extensive experiments that show the action of current meters under extreme conditions, as well as the conditions ordinarily met with in practice, and are to be commended for their work. Their summary of the comment of other investigators on the behavior of various types of current meters under different conditions, is also valuable.

The authors mention that the meters used in the experiments were not rated at the time of the tests because only the relative values of the observations when compared among themselves were to be considered in the experiments. In Table 2, however, the velocities indicated by the meters are compared with the velocity computed from the weir discharges. Obviously, in this case, any error in the meter rating will be included in the deviation due to the obstruction in the channel. The error due to the inaccuracy in the rating of the meter is probably small in comparison with the differences observed; nevertheless, there is a definite change in the rating of meters as is shown by Table 4[6] which gives the ratings of a small Price electric meter extending over the period from 1919 to 1929. All these ratings were made under the same conditions at the station of the Hydraulic Laboratory at Fort Collins, Colo. It will be noted that, in general, the changes are small from year to year, but occasionally there is a definite change. This is not due to inaccuracies inherent in rating, because the check ratings in 1921 and 1929 are almost identical.

Almost without exception the variation in mean velocity caused by different obstructions in Table 2 is greater at the low velocity than at the high velocity. This seems unusual. Ordinarily, the faster the water flows, the greater is the disturbance caused by the obstruction. In the data shown in Table 2 this seems to hold true only in the case of the submerged weir.

The data plotted in Fig. 12 and Fig. 13 show that under some of the extreme conditions of the test, single-count meters give very erroneous results. This does not mean that errors of this magnitude are to be expected under conditions met with in practice. It is doubtful if the velocity would ever reverse at a point in any section that would be chosen for a current-meter measurement. It is true that, at some rating stations, the water may flow backward near the banks at certain stages, but as long as it remains at the same stage, it will continue to flow in the same direction. The records of all the meters show that a variation of 50% in the velocity has practically no effect on the accuracy of registration.

---

[5] Assoc. Irrig. Engr., Colorado Experiment Station, Fort Collins, Colo.

[6] Unpublished records of the Irrigation Investigations of the Div. of Agricultural Eng., Bureau of Public Roads, U. S. Dept. of Agriculture, co-operating with the Colorado Experiment Station.

TABLE 4.—RATING OF A SMALL PRICE ELECTRIC METER MADE DURING THE PERIOD FROM 1919 TO 1929, AT THE HYDRAULIC LABORATORY, FORT COLLINS, COLO.

Date.	Equation.	Remarks.
June 7, 1919	$V = 2.201\ R + 0.025$	....
November 24, 1919	$V = 2.214\ R + 0.020$	....
November 26, 1920	$V = 2.199\ R + 0.052$	....
July 5, 1921	$V = 2.185\ R + 0.022$	Check ratings.
July 5, 1921	$V = 2.185\ R + 0.026$	
September 27, 1921	$V = 2.176\ R + 0.027$	....
March 3, 1922	$V = 2.183\ R + 0.040$	....
March 31, 1923	$V = 2.196\ R + 0.035$	....
June 16, 1924	$V = 2.198\ R + 0.020$	New shaft installed.
May 28, 1926	$V = 2.330\ R + 0.050$	Velocities less than 2.4 ft. per sec.
May 28, 1927	$V = 2.172\ R + 0.023$	New pivot and shaft installed.
April 16, 1928	$V = 2.166\ R + 0.048$	....
July 15, 1929	$V = 2.172\ R + 0.031$	Check ratings.
October 18, 1929	$V = 2.173\ R + 0.030$	

As shown in Table 3 none of the meters, except the small Ott, gives very satisfactory results when measuring the axial component of the velocity if the water approaches the meter at an angle to the horizonal plane. All the Price meters over-register, because they measure the velocity in the direction that the water is moving and not along the axis of the meter. The errors in the Price meters shown in Table 3 are the deviations due to multiplying the axial velocity by the cosines of the angle at which the water approaches the meter.

TABLE 5.—EFFECT OF MULTIPLYING THE AXIAL VELOCITY BY THE COSINES OF THE ANGLES AT WHICH THE WATER APPROACHES THE METER.

Description.	Type of meter.	HORIZONTAL ANGLES, IN DEGREES.						
		10	20	25	30	35	40	45
Reduction in velocity, in percentage, due to multiplying by cosine of angle.	(Computation)	1.5	6.0	9.4	13.4	10.1	23.4	29.3
Over-registration of meter, in percentage, when compared with axial component of velocity.	Price Meter, No. 211 880	1.4	4.3	8.7	12.8	19.5	26.6	36.9
	Price Acoustic	0.7	4.7	8.0	12.4	18.2	25.2	34.8
	U. S. Geological Survey Improved Price	1.0	5.0	6.8	11.7	18.0	25.4	36.2

This is proved by Table 5 which was derived from Table 3, and shows for each angle and each Price meter the reduction of the velocity caused by multiplying by the cosine of the angle and the over-registration of the meter when compared with the axial component of the velocity. In the comparison, the means of the right and the left deviations are used.

As shown by Table 5, the horizontal angle at which the water strikes the meter apparently has very little effect on the accuracy of registration of the velocity in the direction of flow by the Price meters until it exceeds 40 degrees.

TABLE 6.—COMPARISON OF CURRENT METER MEASUREMENTS BY DIFFERENT METERS AND METHODS WITH WEIR DISCHARGE WHEN CURRENT METER IS TURNED 10° TO THE RIGHT BOTH BEFORE AND AFTER WEIR DISCHARGE WAS REDUCED BY MULTIPLYING BY THE COSINE OF 10 DEGREES.

Meter.	Flume width, in feet.	Depth in flume, in feet.	Mean velocity, in feet per second.	Weir discharge, in cubic feet per second.	DEVIATION FROM WEIR DISCHARGE.							
					Method of Current Meter Measurement.							
					Vertical Integration.		Multi-Point.		Two and Eight-Tenths.		Six-Tenths.	
					With cosine correction, percentage.	Without cosine correction, percentage.	With cosine correction, percentage.	Without cosine correction, percentage.	With cosine correction, percentage.	Without cosine correction, percentage.	With cosine correction, percentage.	Without cosine correction, percentage.
Small Price electric, with tail	7.980	1.742	1.076	14.96	+1.15	−0.38	......	......	......	......	......	......
	7.980	1.742	1.079	15.00	......	......	+1.35	−0.13	+1.56	+0.07	+5.62	+4.07
Small Price electric, without tail	7.982	2.118	1.796	30.36	−0.27	−1.78	......	......	......	......	......	......
	7.982	2.115	1.789	30.21	......	......	0.00	−1.46	+2.79	+1.29	+4.27	+2.74
Hoff experimental, with tail	7.981	1.963	1.516	23.75	−3.25	−4.72	......	......	......	......	......	......
	7.981	1.963	1.516	23.75	......	......	−1.87	−2.86	−0.94	−2.24	−2.31	−3.79
Small Ott, with tail	7.982	2.102	1.772	29.74	+1.12	+1.65	......	......	......	......	......	......
	7.982	2.091	1.751	29.22	......	......	+1.21	−0.24	+1.60	+0.14	+5.07	+3.56

This characteristic of the Price meters has its advantages, as well as its disadvantages, because for this reason it is not necessary to hold the meter exactly parallel to the axis of the stream in order to make it register accurately. This is not true of the propeller meters.

When the water approaches the meters at a small horizontal angle, all the meters register the axial component of the velocity with a fair degree of accuracy, as shown by Table 3. This is also shown by Table 6 which gives the results of some experiments[7] on cup and propeller meters using different methods of measurement made at the Bellvue Hydraulic Laboratory in 1921. In making these tests, the water was measured by the current meters in an 8-ft. rectangular channel and checked by a standard 10-ft. Francis weir constructed according to the plans of the weir used in the Lowell hydraulic experiments. Each meter was held so that its horizontal axis made an angle of 10° with the axis of the channel. The results given in Table 6 show the deviation in the measured discharge by each meter when compared with weir discharge, both with and without the cosine correction. The table shows that the results of the measurements with the small Price electric with tail and the small Ott with tail are closer without the cosine correction; the results for the Hoff meter are bettered by introducing the cosine correction, and the results for the small Price electric without tail are divided.

---

[7] Unpublished records of the Irrigation Investgations of the Div. of Agricultural Eng., Bureau of Public Roads, U. S. Dept. of Agriculture, co-operating with the Colorado Experiment Station.

All the tests reported by the authors show that small changes from the normal cause relatively small errors in the registration of the meters. The large errors occur when the meters are subjected to unusual conditions. The tests indicate that, if the rating station is chosen properly, both the cup and the propeller meters will indicate the velocity with a satisfactory degree of accuracy.

Brehon B. Somervell,[8] M. Am. Soc. C. E. (by letter).—The authors have added some valuable and interesting information to the data available as to the behavior of certain types of current meters under conditions differing from the regular and normal flow of the water measured. The Board on Sand Movement and Beach Erosion appointed by the Chief of Engineers of the Army to investigate conditions at points along the Atlantic and Gulf Coasts, was faced at the beginning of its investigation with the necessity for finding a meter which would not only give reasonably satisfactory measurements with varied flow, but one which would record reversals in flow and currents of very low velocity. In the investigation of wave action on the coast it was necessary to determine certain characteristics of the waves themselves and to ascertain the combined effect of the currents caused by waves, winds, and tides.

After a canvass of the experience of the members of the Board and its Consultants, and after conferences with other Government agencies having extensive experience with current meters, the conclusions were reached that (1) there was no meter on the market that would record reversals of current every few seconds; and (2) none that would record the extremely small velocities which were expected to be found when all forces acting on the water except those of the tides were negligible. The service which the meter was required to perform evidently demanded an horizontal-axis apparatus because one with a vertical axis was not adapted to measure the direction of flow of the water near the ocean beach. Various types of horizontal-axis meters manufactured in the United States as well as a French and a German meter were considered; some of them were actually put in use, to determine to what extent they might serve to measure the currents encountered. In the Ott current meter, reversals in current were shown by the alternate occurrences of long and short rings of a buzzer, denoting three and two revolutions, respectively. Inasmuch as these revolutions sometimes were very rapid and sometimes extremely slow, it was impossible to tell from the recording device whether the rings were short or long. For this reason the direction of rotation could not be established definitely, except in shallow water where the motion of the propeller could be observed. All were found to be inadequate by reason of the fact that they could not record reversals in the currents for this and other defects.

The essential parts of the meter, as designed by Dean G. B. Pegram, of Columbia University, are shown in Figs. 18, 19, and 20. The inertia of the

---

[8] Maj., Corps of Engrs., U. S. A., Memphis, Tenn.

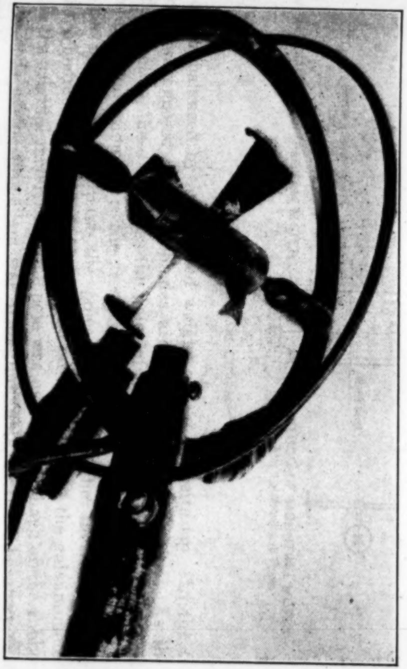


FIG. 18.—DETAILS OF PEGRAM CURRENT METER.

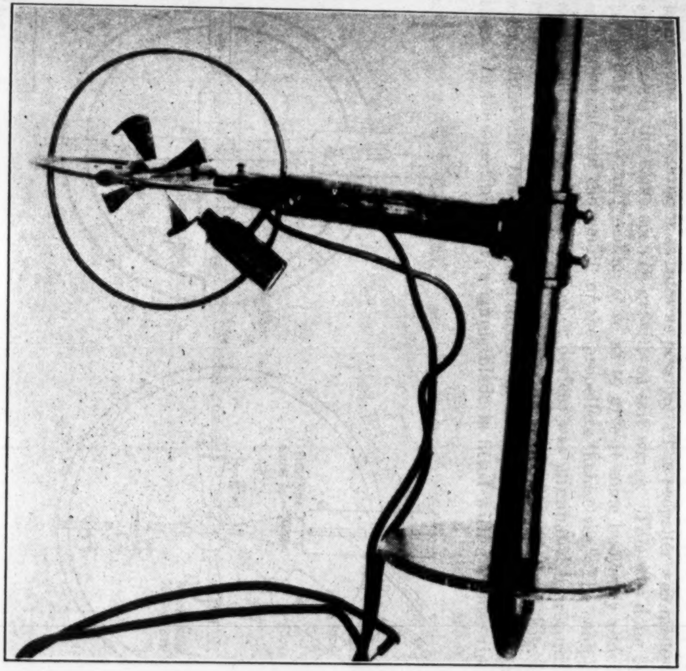


FIG. 19.—MOUNTING OF PEGRAM CURRENT METER.

moving parts is overcome to a great extent by mounting the propeller in glass bearings and by making it as light as possible. The propeller was made to have approximately the same specific gravity as salt water. This was done by making the hub hollow and sealing it so as to give it some buoyancy which overcame its weight and the weight of the propellers themselves. The complication of additional gears, shafts, and other devices was eliminated by recording the revolutions of the propeller electrically. As will be noted from Fig. 20, the meter is fitted with two poles insulated from the frame. The propeller has four blades. In the one-tip meter, a single blade is fitted with a rubber tip which

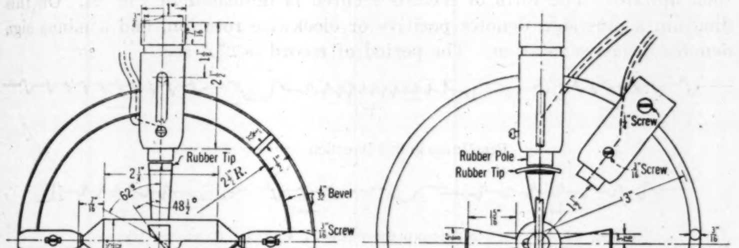

1"
2"
Rubber Tip
2¼"
2½" R.
62°
48½°
Bevel
Screw
Propeller Spindle
Hollow
Rubber Tip
Screw
Rubber Pole
Rubber Tip
Screw
3"
Propeller Blades
28 Gauge Brass
48 Volt Battery
Weight 140 Pounds
A
Rheostat
B
G
Recording Galvanometer
"Direct Writer"
Water Level
WIRING DIAGRAM OF PEGRAM METER
Ground
M₁
M₂
METHOD OF MOUNTING METER
Weight of Mounting 75 Pounds

FIG. 20.—PEGRAM CURRENT METER.

is counterbalanced by an opposed weight on the opposite tip. In the two-tip meter, two opposed propeller blades are fitted with rubber tips. As the propeller revolves, these tips pass the poles of the meter which are connected to a recording galvanometer; the resistance of the sea water is cut down as the insulated tips pass the poles, and, with the increase in conductivity, a characteristic mark is obtained on the record of the galvanometer.

In the original meter, only one pole was used. The shape of the rubber tips on the ends of the propellers was triangular and, in the laboratory, a characteristic notch was secured on the recorder when the blunt end of the tip passed the pole first; when the sharp end of the tip passed first a different notch was recorded. However, when this meter was used in the field, it became very difficult to distinguish between the notches caused by rotations in one direction and those caused by reversal of direction. To obviate this uncertainty, an additional pole was placed on the frame of the meter. The result of adding this pole was to cause a double deflection as the tip passed the two poles. If the direction of rotation of the propeller was positive, the recorder was first deflected upward and then immediately downward, whereas, were the direction of rotation negative, the deflection was first downward and then upward. The form of records secured is indicated in Fig. 21. On this diagram a plus sign denotes positive or clockwise rotation, and a minus sign denotes negative rotation. The period of record is 27.5 sec.

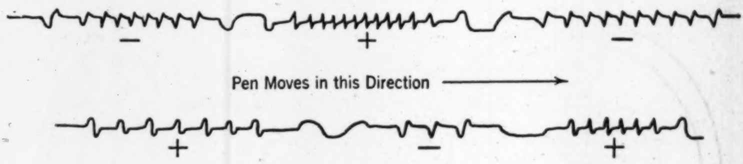
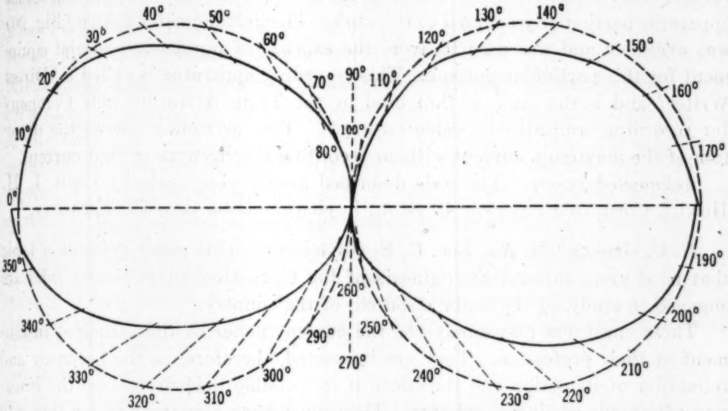


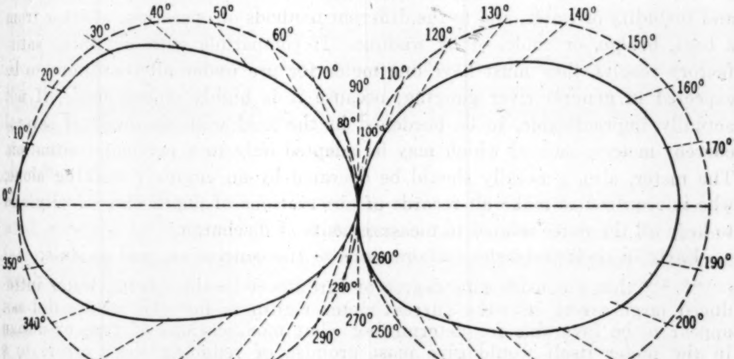
Fig. 21.—Form of Record Secured by Pegram Current Meter.

In so far as recording reversals of current is concerned, the meter has been successful. Its sensitivity has likewise been all that could be desired, readings as low as 0.08 ft. per sec. being secured. Obviously, the two-tip meter is better adapted to measuring small velocities than the single-tip apparatus. With the two-tip meter, half revolutions are recorded; hence, a reversal of one-half a revolution will be shown by the recorder. From the rating curves, this is equivalent to a reversal of 3 in. This is much more sensitive than any other meter at present in use.

In tests made by holding the meter in a stream of water moving through a trough and by moving the meter on a carriage through a swimming pool, certain defects became apparent. Although both these tests left something to be desired in the way of absolute precision of measurement, they have shown that the meter is applicable to the purposes for which it was designed, and that in the measurement of oblique currents, such as might be encountered in turbulent water, its accuracy is at least as great if not greater than those described by the authors. A graphical representation of the results secured in some swimming pool tests is given in Fig. 22. When inclined up to 20° in a positive direction, the Pegram meter gave little or no variation from the cosine curve; at 20° in a negative direction from 5 to 15% variation; at 40°, the meter under-registered in a positive direction about 15%, and at 40° in a negative direction, it under-registered 20 to 30 per cent. Considering the motion in a positive direction only, the results appear to be more satisfactory than those obtained with the meters used in the authors' experiments. These tests seem to indicate further that some change in the shape of the blades of the propeller would be desirable, in which case equally accurate readings in both positive and negative directions could be secured. In the tests made by the Board on Sand Movement and Beach Erosion, two meters were mounted on a staff, at right angles to each other, and the components in the two directions were measured. In this way, the final result becomes an error in direction and a still smaller error in velocity, since the combined velocity becomes the square root of the sum of the squares of the individual velocities.



(a) ONE-TIP PEGRAM METER



(b) TWO-TIP PEGRAM METER

FIG. 22.—RESULTS SECURED IN THE SWIMMING POOL TESTS.

The passage of the water through the meters was retarded to some extent by the mounting adopted. To reduce the size of the mounting, however, would probably result in a loss of strength which would prevent the use of the meter in any but the very mildest seas. The accuracy of the meter can also be somewhat improved by a better balancing of the propellers. This must be done under water, as the difference in the specific gravity of the rubber and brass in the propeller causes the rotating parts to become unbalanced when they are placed in the water. With these improvements the meter should be of considerable value in salt water, or in liquids having some conductivity. At present, it is somewhat cumbersome because of the weight of the recording apparatus. This weight can be easily reduced, however, by substituting other apparatus particularly adapted to the work. The galvanometer device (Fig. 20) was available and was used to avoid the expense of purchasing special equipment for this particular purpose. This recording apparatus is called a "Direct Writer" and is the same as that used by the Trans-Atlantic Cable Company for recording transatlantic cable messages. The instrument shows the direction of the maximum current without recording the strength of that current.

Acknowledgments.—The tests described herein were made by Lieut. L. H. Hewitt, Corps of Engineers, U. S. A., in charge of the Board's field party.

N. C. GROVER,[9] M. AM. SOC. C. E. (by letter).—This paper presents a topic that is of great interest to engineers of the U. S. Geological Survey who are engaged in studying the water resources of the country.

These engineers necessarily use the current meter as the principal instrument of their profession. They are interested, therefore, in the accuracy and reliability of its action and the effect of its possible inaccuracies on the integrity of records of river discharge. They must have a current meter that will give reliable results under a wide range of conditions related to velocity, depth, and turbidity of water, and to the different methods of operation, whether from a boat, bridge, or cable, or by wading. If compatible with obtaining satisfactory results, they must have one meter for use under all conditions to be expected in general river gauging, because it is highly objectionable, if not actually impracticable, to be burdened in the field with an outfit of several current meters, each of which may be adapted only to a particular situation. The meter, also, generally should be operated by an engineer working alone, who is required to make all records of observations of depth and velocity and to keep all the notes related to measurements of discharge.

Early in their tests, the authors came to the conclusion, and so state,

> "* * * that a considerable degree of turbulence in the flowing water introduced large errors into the current-meter readings; but the results did not appear to be favorable for determining what modifications of type or design in the meter itself would give most promise of reducing these errors to a minimum. Furthermore, multiplying the number of such experiments did not seem to be a worthy procedure, largely because there is no existing method of measuring degree of turbulence and no standards or units in which turbulence may be expressed."

The conclusion thus reached is indicative of the difficulties experienced by field engineers as well as by laboratory investigators in connection with the measurement of turbulent water and the development of a current meter that will reliably measure such water.

Just as engineers have recognized that good leveling work can not be done in unsteady air, so they have recognized that good current-meter work can not be done in turbulent water, and as they have learned to refrain from attempting to run precise levels under conditions of unsteadiness, so they should learn that they can not do first-class current-meter work in turbulent water by using any type of current meter yet devised. The greater the unsteadiness or turbulence of either air or water the greater will be the possible inaccuracies in the measurements. Unfortunately, it is impossible to define the degree of unsteadiness of either the vibrating air or the turbulent water and, as far as the writer knows, no method has been devised for expressing even approximately the different degrees of turbulence as a basis for the study of inaccuracies or of corrections for them. Thus, the authors have brought again to the attention of engineers the basic facts related to the varying effects of turbulence on meter action, but they have not attempted to define or describe the different degrees of natural turbulence of water in such manner as to form a basis for corrections to be applied to current-meter observations.

---

[9] Chf. Hydr. Engr., U. S. Geological Survey, Washington, D. C.

The authors state that,

> "While the data of Table 1 do not disclose the correct values of the average velocity of the current, it became evident from the angularity tests made later that the Price meter recorded too high a value and that the values obtained with the other meters are in varying degrees below the true amount."

and in their conclusions that,

> "When meters are held on a rod in a rigid position, with the meter axis parallel to the general axis of the stream, turbulent flow invariably causes the cup meter to over-register and the screw type to under-register."

Then, they add the following words of caution:

> "It should be remembered also that rod support was used in all these experiments, and the conclusions must not be applied without modification to cable-suspended meters. In a majority of cases, stream gauging is done with cable-suspended meters. In such suspension the tail-vanes tend to keep the meter squarely facing the current regardless of any local or temporary obliquity of flow."

In these statements, the authors bring out clearly the kind of suspension used, whether by rod or cable (a condition often overlooked by laboratory investigators) and, in addition, they state what is perhaps the most important result of their tests. Although it has long been recognized that cup-type current meters over-register under conditions of considerable turbulence, it has not been so generally recognized that propeller-type meters under-register by comparable amounts. The authors have made a valuable contribution to engineering literature in making clear this important fact.

As engineers of the U. S. Geological Survey must use cable suspension in connection with a large proportion of their measurements, a comparison between the behavior of propeller-type meters and cup-type meters on cable suspension would be very desirable. However, this comparison remains now, as before, entirely unknown. It may be reasonable to suppose, as the authors have inferred, that, with all types of meter, obliquity of current may not be so important a factor with cable suspension as with rod suspension.

There is a belief apparently widely prevalent, especially among engineers of limited field experience and laboratory workers who are likely to overlook the important aspects affecting field serviceability of the meter, that the propeller (horizontal-axis) type of meter is superior in every way to the cup (vertical-axis) type of meter. Whatever the theoretical advantages of the former, however, the latter seems to have certain practical aspects of superiority that have been controlling. These are related to delicacy of action, essential accuracy, adaptability to use under a variety of conditions, sturdiness, lightness, and reliability. The authors limit their discussion to accuracy of registration. They have referred to these other aspects in a pertinent way in the final paragraph of the paper.

The development of a current meter so that it may serve with even reasonable satisfaction under the wide variety of conditions to be found in general river measurement work is difficult. The small Price meter has perhaps been used more extensively and has received more expert study than any other current meter. As a result, in part, of this situation and, in part, of the natural advantages of the cup type, the small Price meter is now better adapted to general use than any other meter that has been developed. It is light and yet strong; sensitive and yet durable; it will measure, without great error, velocities ranging from less than ¼ ft. per sec. to velocities sixty times as great. It can be used readily whether suspended by a cable or supported on a rod. It will hold its rating under rough usage; it is easily repaired, is quickly taken apart from cleaning and oiling, and is re-assembled just as quickly without change in rating. It is sensitive, although it is not equipped with ball bearings. It can be used by an engineer working without an assistant. No other current meter known to engineers of the U. S. Geological Survey satisfies these conditions.

On the other hand, this meter generally over-registers in turbulent water. Its lack of adaptability to such conditions should be recognized, and it should not be used in eddying or seething water or from a boat that, by a rolling motion or by rising and falling on waves, gives the meter a churning motion.

Many attempts have been made to develop a propeller type of meter for general use, and engineers look forward with hope to such a meter in the future. The writer is informally advised that engineers of the Dominion Water Power and Reclamation Service of Canada, starting with an Ott meter, are about to produce their fourth trial meter of the propeller type to meet their needs in situations where there were difficulties with frazil ice which filled the cups of the small Price meter. They are to be congratulated in their attempts and should be encouraged to continue until they have produced a satisfactory propeller type of meter.

There are, however, many difficulties to be overcome before the equal of the small Price meter can be produced. The practically frictionless pivot bearing of the vertical-axis type of meter can not be used with the propeller type; cylindrical bearings must be substituted, if the advantages to be obtained from the overhung propeller are to be realized. Ball bearings are immediately suggested, but it is still an open question whether such bearings can be made as nearly frictionless as pivot bearings of the vertical-axis type. This will be apparent especially when it is recognized: (1) That the meters must be used in silt-laden rivers at depths greater than 50 ft.; (2) that exclusion of water from the bearings under these conditions is probably impossible; (3) that silt, often gritty, will be forced into them to interfere with the action of, and to increase the friction in, the bearings; and (4) that such bearings are not easily cleaned. In addition, the propeller must be overhung if there is to be no obstruction in front of it. This, of course, entails the use of a shaft of considerable strength and size, thereby increasing the friction and decreasing the sensitiveness of operation of the meter. The wheel must also be perfectly balanced and must remain so under ordinary usage, or it will not operate evenly in water that moves at small velocity, but will tend to stop when the heavy part of the propeller is below the axis. By contrast, a meter of the cup type, equally eccentric on its axis, would continue to revolve in water moving at the same velocity. Even a minor injury to the vanes of a propeller type of meter will change the rating, whereas experience has shown that injured cups of the small Price meter can be pressed into approximate shape in the field and the use of the meter continued without change of rating.

Because of inaccuracies in the operation of current meters in turbulent water, engineers for many years have specified that sites for river-gauging stations must be so selected that current-meter measurements will be made in water that is free from cross-currents, boils, and eddies; and, ordinarily, river measurement work is performed under selected conditions that are known to give reliable results. This situation is recognized by the authors in their reference to "the good velocity conditions usually found at most gauging stations." That the results obtained by engineers of the U. S. Geological Survey are generally reliable has been shown many times by comparisons with measurements of discharge made by other current meters and in other ways.

The latest comparisons of this kind that have come to the attention of the writer are the following:

A comparison between measurements by means of the salt-velocity method and the small Price current meter was made in a canal at Bellows Falls, Vt., on August 16 and 17, 1929. The salt-velocity determinations were made by C. M. Allen, M. Am. Soc. C. E., and the current-meter measurements, by H. B. Kinnison, Assoc. M. Am. Soc. C. E. This statement relative to the comparison has been abstracted from an unpublished report by Mr. Kinnison. The width of the canal at the surface is 100 ft., and at the bottom, 35 ft.; the side slopes are 1 : 1¼; and the depth of water was about 28 ft. The maximum velocity of water observed during the tests was 6.56 ft. per sec. The current-meter measurements were made within the stretch in which the salt-velocity determination was made and about 35 ft. down stream from the pipes used to inject the salt water. There was, therefore, some turbulence in the section where the current-meter measurements were made, due to the pipes. Two salt-velocity determinations were made by Professor Allen in connection with each current-meter measurement of discharge, one at the beginning and one at the end of each measurement. The discharge by the salt-velocity determination is the average of the two determinations thus made. The first comparison was made on August 16 when 1 hour and 41 min. were required for the current-meter measurement. The average of the two salt-velocity determinations was 9 780 sec-ft.; the discharge as determined by current meter was 9 896 sec-ft.; the excess shown by the current meter over the salt-velocity method was 1.19 per cent.

The second comparison was made on August 17 when 1 hour and 18 min. was used in making the current-meter measurement. The mean of the two determinations by the salt-velocity method was 8 587 sec-ft., and the discharge as determined by the current meter was 8 612 sec-ft. The excess shown by the current-meter over the salt-velocity method was 0.29 per cent.

A comparison of measurements by the salt-velocity method and by the small Price current meter was made in 1924 by S. B. Soule, District Engineer, U. S. Geological Survey, Madison, Wis. This comparison was made in the Mississippi River at St. Paul, Minn., between gate-openings of the Ford Power Plant. The gate-openings were rated by Professor Allen by the salt-velocity method. Current-meter measurements were made by Mr. Soule in the river about 1 200 ft. down stream from the power plant. The mean depth of water in the measuring section was about 6.5 ft. Two comparisons were made on December 19, and two on December 20, 1924. In the first comparison the discharge obtained from the gate-openings was 1 927 cu. ft. per sec., and by the current meter, 1 975 cu. ft. per sec.; the excess by current meter was 2.5 per cent. In the second comparison, the discharge from gate-openings was 2 290 cu. ft. per sec., and by current meter, 2 342 cu. ft. per sec., an excess by current meter of 2.3 per cent. In the third comparison, the discharge by gate-openings was 1 990 cu. ft. per sec., and by current meter, 2 033 cu. ft. per sec., an excess by current meter of 2.2 per cent. In the fourth comparison, the discharge by gate-openings was 1 668 cu. ft. per sec., and by current meter, 1 666 cu. ft. per sec., an excess by current meter of 0.1 per cent. The report of these comparisons has been abstracted from an unpublished report by Mr. Soule.

Comparisons have been made by Mr. Kinnison between calibrated weir measurements at the Alden Laboratory, Worcester, Mass., and the small Price current meter as follows:

(1) Discharge by weir, 1.74 cu. ft. per sec.; by current meter, 1.75 cu. ft. per sec.; an excess for the current meter of 0.57 per cent.

(2) Discharge by weir, 25.05 cu. ft. per sec.; by current meter, 24.83 cu. ft. per sec.; a deficiency for the current meter of 0.88 per cent.

(3) Discharge by weir, 31.469 cu. ft. per sec.; and by current meter, 30.83 cu. ft. per sec.; a deficiency for the current meter of 2.0 per cent.

It would be unfortunate if engineers were led by this paper to lose confidence in the reliability of the current meter because, in the writer's opinion, there is no occasion for loss of confidence. The authors have created artificial turbulence in order to show clearly the dangers resulting from the use of current meters in turbulent water and to compare the behavior of meters of different types. They should perhaps place greater emphasis than they have on the fact that their observations of meter operation in water that is not turbulent confirm the essential accuracy of the action of current meters of both standard types when operated under proper conditions.

Moreover, there is no other method for obtaining reliable records of discharge as expeditiously or as cheaply as by use of the current meter. Other methods, which may be used in tests of water-wheels, or otherwise, where accuracy is essential and where a stretch of smooth-flowing water is not available for current-meter work, are not adapted to general river measurement because they are often more difficult to apply and always much more costly in their application.

Burke L. Bigwood,[10] Esq. (by letter).—The experimental data presented in this paper apparently prove conclusively that, under certain conditions of flow, results of stream measurements that are obtained by use of the current meter may be grossly in error. Even without such experimental proof, trained users of the current meter would be cognizant of the existence of such inaccuracies in its operation. The concrete evidence submitted by the authors confirms beliefs heretofore based on general observation and reasoning only.

The current meter is in widespread use at present (1930), as it has been for many years. In the United States and in Canada the cup-type meter is generally preferred. The U. S. Geological Survey uses the small Price meter almost exclusively in its vastly extensive activities in the field of general stream-gauging work.

It is the general opinion that at a first-class river-gauging station this type of meter produces an accurate record of the flow of the stream. This is assuredly the opinion of the personnel of the U. S. Geological Survey. Practical tests have been made which bear out this belief. This meter has been tested against accurate weirs of all kinds, against volumetric methods of measurement, against Venturi meters, and against salt-velocity and other so-called chemical methods of measurement. An imposing array of evidence could undoubtedly be assembled to prove that this meter, properly operated, gives results well within required limits of accuracy. This precision in results has been brought about and maintained by the practical elimination of those conditions of flow which produce the errors in operation of the meter.

First-class river-gauging stations contain as a most essential element a "smooth" measuring section. By this is meant a section of the stream channel through which the water flows with a minimum of turbulence and horizontal angularity. It would be expected that any type of meter would produce good results at such a station, but it is the writer's opinion that the Price meter has one great advantage, indicated by the results of the authors' experiments.

Through the selection of "smooth" measuring sections the adverse effect of turbulence and of lines of flow approaching the meter at angles from the right or left has been practically eliminated; but owing to the character of river beds in general, it is impossible to eliminate the effect of roughness of the bed of the stream on an observation to determine the velocity near the bottom. It is the writer's opinion that the usual rocky or boulder-strewn bed creates waves in the lines of flow near the bottom of the stream. This does not amount to turbulence since lines of flow above, toward the surface, are not disturbed by this action. A meter suspended in water flowing in this manner is acted upon by lines of flow approaching at angles from above or below. Under this condition, the experiments outlined in the paper (Fig. 16) prove that the Price meter gives the best results.

---

[10] Dist. Engr., U. S. Geological Survey, Hartford, Conn.

All meters give least reliable results under conditions of turbulence and horizontal angularity. Having eliminated these conditions by the proper selection of gauging sections, the remaining conditions are such that, in the writer's opinion, the Price meter gives the most reliable results, mainly owing to its ability to measure accurately the wave flow near the bottom of the section.

G. H. Nettleton,[11] Esq. (by letter).—This paper covers important ground from the meter user's point of view. The experimenters are to be complimented on the thorough manner in which the tests were carried out, but the writer cannot agree entirely with their findings for reasons here given.

The fact that the bucket type of current-meters has a tendency to over-register and those of the screw pattern to under-register when they are acted upon by very turbulent streams of water, has been long recognized. In this paper much interesting work has been described that bears out this contention; but it appears that, during the actual tests, meters were subjected only to turbulence of a most violent nature. Such conditions may be exaggerated with the definite purpose in view of ascertaining some weakness in the behavior of an instrument, but the fact must be borne in mind that conditions may be exaggerated to such an extent that they bear little or no relation to actual stream conditions at average measuring sections. This latter contention would appear to be the case in the experiments described by the authors. The turbulence was of such a degree that meters were forced to operate under conditions for which they were not designed and which would not be encountered at what may be described as even the more poorly selected measuring sections. Further, the artificially produced turbulence was of such a high magnitude that it was out of proportion to the mean velocity of the stream, namely, from 1 ft. to 5 ft. per sec.

It is difficult to conceive any conditions which might exist in the open channel of a river having much greater velocities than those used in the tests, which would produce such abnormal turbulence, and it would be even less true of a selected measuring section. Rapids or water-falls might be responsible for such violent agitation and they would cause vortices, etc., but no engineer would ever consider attempting a measurement in a stream at such locations. In support of the writer's contention that the artificially produced turbulence and the conditions under which the meters were used in the tests were not applicable to actual field measurements, he would draw attention to the following points.

The fixed paddle-wheels (Fig. 11) were apparently only 6 ft. up stream from the meter and were of such a size that they actually masked it to a considerable extent, and must have almost, if not entirely, prevented the natural tendency of an unobstructed flow to smooth out deviating threads of current.

---

[11] In Chg., Dominion Govt. Current Meter Rating Station, Dominion Water Power and Reclamation Service, Calgary, Alberta, Canada.

In this case the angle at which the water was caused to impinge on the meters was very much in excess of that encountered in any river, and unobstructed flow could hardly have existed anywhere in the cross-section of the experimental flume at the meter section. Further, in the test with the vertical paddles (Table 1), there was a gap of 3 in. between their bottom edges and the bed of the flume. In the case of one paddle this would have the effect of creating a high velocity undershot current which would turn sharply upward from the flume bed owing to the low pressure area at the back of that paddle. From the viewpoint of producing violent turbulence, the arrangement was excellent, but it would appear impossible to believe that such conditions existed at measuring sections.

Again, rolling vortices appear to be treated as though they were moving along and resting on the bed of a river in the same manner in which a wheel rolls over a road. In fact, in some tests, what may be described as the negative or low velocity side of such a rolling body of water was apparently given a value approximately equal to the mean velocity with which the water was flowing down the flume. This is anything but true of river conditions because such a body of water as a whole is moving down stream and the negative or low velocity side has a forward and not an up-stream velocity in an open channel. These changes in velocity affect the meter in a river as current pulsations and, as is conceded by the authors, pulsations in the flow of a stream do not cause a meter to register incorrectly. It is true that in the experiments pulsating currents did cause some difference in the recording of instruments of the cup and screw type, but the difference was accounted for by the artificially produced effect of the current changing from a positive to a negative value, or from flowing down stream to up stream.

If a meter was freely suspended by cable in a river-measuring section where such conditions existed, the instrument would naturally face the direction of flow and, consequently, at times, would be pointing up stream and at others, down stream. Such conditions have never been encountered by the writer even in turbulent mountain rivers, but the flowing water always had a down-stream or positive value, even though the velocity might vary from time to time. Experiments carried out by the writer showed that what may be described as pulsating currents, that is, currents the velocity of which rose and fell above and below a mean velocity, did not cause either a meter of the screw or cup type to register incorrectly. The value of the pulsations was exaggerated purposely so that the difference in rise and fall of the velocity was considerably in excess of anything of the nature that had been experienced in field work, but at no time was the velocity given a negative or up-stream value. Furthermore, numerous measurements of different rivers have been made, using a cup and screw type of meter at the same time. These rivers were representative of those found in the Province of Alberta, Canada, in the foot-hills of the Rocky Mountains. In all cases the difference between the mean velocity recorded by each type of instrument was of such small value that it was negligible, and, further, the cup meter did not always show the greater velocity; frequently the screw meter gave the higher figure.

Although it is agreed that violent disturbance of a stream will usually cause a cup type of meter to over-register and a screw pattern to underestimate true velocity values, yet it is considered that at the average selected measuring sections, instruments of either pattern will give reliable data to within the accepted degree of accuracy for stream-measurement work. Furthermore, the experiments, as described in the paper, were conducted under abnormal conditions of flow and turbulence which made it impossible for the current meters to function either as they would in the field, or under the conditions for which they were designed.

C. LINDEN,[12] ESQ. (by letter).—The investigations described by the authors have added valuable data to the information pertaining to current meters and current-meter action; but like all previous tests these have failed to bring definite results or to end the controversies which have prompted the studies.

Investigations are doomed to failure when it is assumed that the forces which actuate the meter represent only the true velocity. Under ideal conditions this may be very nearly true, but in turbulent waters the flow represents a combination of forces of which the velocity may be only a small part. Judging from the design of some well-known meters it must have been assumed that anything which may be rotated by the forces of flowing water may be rated and thus may become a current meter. If streams proceeded as parallel filaments, this assumption would be true, but such is not the case, and meter rotation is not a definite product to which correcting factors may be successfully applied. Meter rotation is the result of forces of irregular origin.

Other erroneous assumptions which have been made are that meter work can be properly done in both clear water and in water filled with floating débris, and that meters should be rated under field conditions similar to those in which they will be required to operate. This would be highly desirable, but it is impractical because of the ever-changing condition of both direction and velocity. No mechanical meter can produce accurate results unless the flow and operating conditions are identical with those under which it was rated. Under all other conditions the results are only approximate. The object of meter studies is, however, to produce an instrument that will approach accuracy as nearly as possible when operated under field conditions. This has proved to be a much more difficult task than was at first anticipated.

It is not possible to establish comparable ratings from meters of various types when simply held in streams of varying turbulence. The importance of maintaining a constant and definite relation between the position of the meter axis and the direction of flow does not seem to have been fully recognized in past investigations.

Theoretically, stream velocities can be measured as a function of the velocity force, but it is too much to assume that true stream velocities can be secured by mechanical means. In order to do so, the mechanical device must be capable of differentiating between true velocity forces and other influences. It is not likely that irregular stream conditions, in which the re-acting forces are caused by the influence of the meter itself, will result in consistent meter action. In turbulent or extremely irregular water, such as is found in the tail-races of power plants, it is too much to expect a meter to distinguish between true velocity forces and the many other disturbing and re-acting forces. Mechanical devices for stream-flow measurements have no place under these conditions, although many investigators apparently depend on records obtained under such conditions for stream flow data.

---

[12] Secy., Scientific Instrument Co., Berkeley, Calif.

Any mechanical device or meter will act in accordance with the forces applied, and identical forces will give identical readings. It has been suggested that meters should be calibrated or rated under field conditions in order to have stream conditions the same as those in which the meter is to be used; but since the calibration of a meter involves the establishment of a true ratio between flow velocity and meter rotation, it presupposes definitely known velocities and a constant flow direction. In the field neither of these essentials is present and, therefore, meter ratings under these conditions are impossible. Current-meter rating consists of more than exposing a meter to stream flow or causing the meter to move through still water. The meter must be held in a stable, least flow-obstructing position during the rating process.

Some stations do not give identical ratings even for the same meter on repeated readings. This is due to limited cross-sections of the rating channel, instability of the meter support, and inaccurate measurement of both time and distance of travel.

The rating flume used by the authors was evidently not of standard dimensions. In order to effect an ideal flow the channel should be not less than 60 ft. long, with a boarded section the full width of the canal. When the flume is less than the full width of the canal, it should exceed 60 ft. in length, in order to overcome, completely, the entrance eddies. A cross-section of 3 by 4 ft. is insufficient even for meters of the small Price type where, under most favorable conditions, the flow-obstructing area is about 4½ in. horizontally and 2¼ in. vertically. With a limited cross-section of the rating flume such an obstruction creates changes from the normal flow condition which affects meter action beyond the allowable limit. Flow conditions become turbulent as the stream filaments come in contact with the meter (which is a flow obstruction) even though the natural flow before submergence of the meter may be ideal.

Turbulence as the result of the presence of the meter in the flow line is objectionable but unavoidable. It is particularly objectionable because its effect on meter rotation is difficult to determine. This factor is, however, subject to some control and, in fact, offers the greatest field for improvement in meters. Improvement may be accomplished by designing a meter rotor so that it engages automatically for its rotation the forward component of the approaching stream forces. An efficient meter must be sensitive; but sensitiveness may be of two kinds—minimum friction and responsiveness to changes in flow direction. In some vertical-axis meters the friction is almost negligible, and yet they are not affected by changes in direction of flow. Such meters appear more conservative when exposed to turbulent flows than actual conditions justify. Although the friction may be slightly greater in horizontal-axis meters, they respond to changes in flow direction so readily as to appear erratic, thus making it impossible to obtain consistent meter indications when the flow is unfavorable to meter work. In neither case does the meter record true flow velocity. The means of testing these features of a meter as described in the paper are unique, but of little practical value. A meter should be as frictionless as possible in order to increase its sensitiveness. As a rule the sensitiveness of a meter is indicated by the lowest velocity at which it starts rotation. Such starting rotation, however (when the ratio between friction and flow velocity is very high), may be caused as much through turbulent forces as by true flow velocity, since these turbulent forces may be the result of the meter's presence in the stream.

The influence of friction in a meter is confined to low velocities and the low velocity curve for a highly sensitive meter is very short. Sensitiveness increases the range of velocities over which the meter is reliable.

Meters cannot be satisfactorily compared unless the tests are made under the most favorable conditions. Ratings made under these conditions bring out information on the efficiency and reliability of a meter. Meters exhibit their characteristics at their low velocity ratings. Accuracy is recognized by the regular distribution of rating points; efficiency, by the shortness of the low velocity curve; and sensitiveness, by the steepness of the slope of the rating curve.

With ideal flow and operating conditions, any meter will act reliably, but to select the proper meter for field conditions one should examine the rating curve carefully. A meter for general purposes, or one which will be required to operate under varying stream or channel conditions, should be of a design having the least flow obstuction; it should have a rotor that will engage the maximum of stream filaments consistent with its size; it should be immune from vertical forces so that it may be used in vertical integration; and, it should have some means of securing a stable position automatically.

Under favorable conditions the records from all meters will agree, but under unfavorable conditions they will differ. This disagreement, however, is no greater than that often obtained from repeated measurements by the same meter. The meter that produces the least interference to flow will produce the most nearly correct readings. To operate properly, it must be held rigidly in such a position in the stream, that it is least obstructing to flow, regardless of the velocity. The meter that presents the least resistance to the flow is most easily held in this position.

H. R. LEACH,[13] M. AM. SOC. C. E. (by letter).—This paper has re-opened the subject of the performance of current meters in turbulent water, a question that has never been satisfactorily settled. It has generally been recognized that, except under conditions closely approaching stream-line flow, current meters of all types may indicate velocities departing more or less from the true velocity of the current, and that as conditions depart further from stream-line motion and turbulence increases, the departures of the indicated from the true velocities become increasingly greater. During the thirty years preceding 1930 several investigators have made excellent observations to determine the magnitude of these departures, and although definite quantitative results have been obtained there has been little, if any, attempt to interpret them so that velocities observed under abnormal conditions could be modified to give more nearly correct results.

---

[13] Prin. Asst. with Robert E. Horton, Voorheesville, N. Y.

In performing these experiments the authors show their recognition of the futility of trying to dispose of a troublesome question by saying, as has been done by many writers on the subject, that current meters should not be used in turbulent water. This merely "sidesteps" a situation that frequently confronts the man engaged in stream measurement, who often has no choice but to use his meter in a turbulent stream or to give up the measurement altogether. Even an admittedly inaccurate measurement may be of value, particularly if some idea of the probable error can be obtained.

In attempting to solve this problem the authors first ran a series of experiments on meters in turbulent water, but found too many factors involved to permit a clear interpretation of the results. Therefore, "rather than leave the matter in this incomplete and unsatisfactory state, it appeared desirable to attempt to resolve the problem into simpler elements and to attack them separately." Accordingly, they performed two other series of experiments. They did not, however, follow up the results obtained in the later experiments and attempt to interpret the performance of the meters in turbulent water in the light of the results found in their subsequent work. This is the only criticism that the writer makes of an otherwise excellent account of well planned and carefully performed experiments. It is to be hoped that the discussions of this paper and the authors' closure will analyze these experiments and suggest some rational method whereby current-meter measurements, that must of necessity be made in turbulent water, can be corrected to give a fairly close value of the true discharge.

As far as the writer is aware, B. F. Groat, M. Am. Soc. C. E., is the only investigator who has suggested a definite method of correcting current-meter measurements made in turbulent water.[14] In connection with measurements of the discharge through turbines at Massena, N. Y., he used Price and Haskell meters and suggested that the true velocity be taken as that indicated by the Haskell meter plus one-seventh of the difference of the velocities indicated by the Price and the Haskell meters.

Experiments on Current Meters by B. F. Groat.—A few years after he had made this suggestion, Mr. Groat ran a series of tests in connection with a comparison of current-meter measurements with chemical gauging.[15] These covered a wide range of conditions. The writer has recomputed these experiments and the results are presented in this discussion to show to what extent they corroborate those of the authors.

The tests were made in the Naval Tank of the University of Michigan on a small Price meter, two low-pitch Haskell meters, and two Ott meters. The meters, with tails removed, were rigidly attached to arms on the rating car with their axes parallel to the axis of the tank and to the direction of motion of the car. The arms were arranged so that they could be oscillated in three directions, that is, vertically, horizontally at right angles to the axis of the tank, and longitudinally parallel to the axis of the tank. They were connected so that the same movement was imparted to each of the five meters. Their axes were always parallel to the direction of motion of the car except that in the longitudinal oscillations the arms gave the meters a slight rocking motion in a vertical plane. The manner in which the frame was oscillated was not stated. Mr. Groat plotted the observed revolutions per foot for different oscillations against the velocity of the car.

---

[14] *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 819.

[15] "Chemi-Hydrometry and Its Application to the Precise Testing of Hydro-Electric Generators," *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1231, Pt. V.

By first constructing a rating curve from the normal observations and then determining the indicated velocity for each observation, the writer obtained the difference between the indicated velocity and the forward velocity of the rating car. The rate of oscillation and the velocity of the car were added vectorially to obtain the absolute velocity of the water relative to the meter and the angle, α, between the direction of the absolute velocity and the axis of the meter. This recomputation was made only for the observations on the Price and Haskell meters.

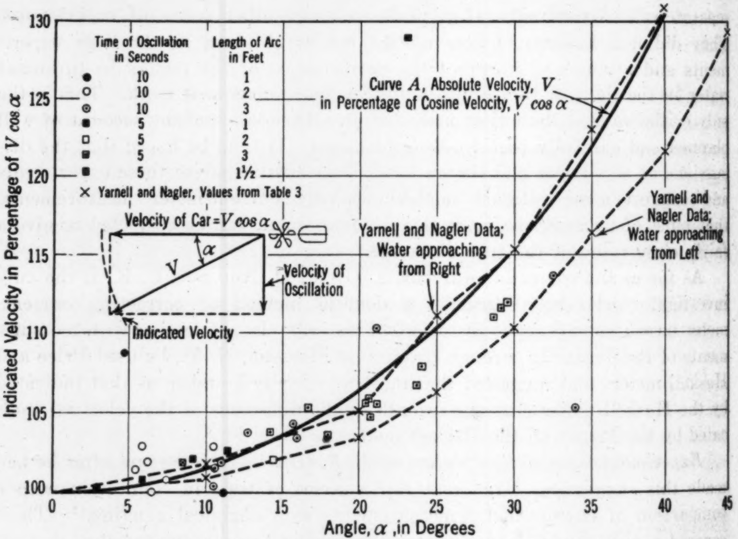


FIG. 23.—HORIZONTAL TRANSVERSE OSCILLATIONS ON SMALL PRICE METER.

Small Price Meter: Horizontal Oscillations.—The observations for horizontal transverse oscillations are plotted on Fig. 23. The indicated velocity in percentage of the cosine velocity is plotted against α, the angle the tangent of which is the velocity of the horizontal oscillation, divided by the velocity of the car. The cosine velocity is the absolute velocity times cos α, and is, of course, the velocity of the car. Curve A is the absolute velocity of the water relative to the meter in percentage of the cosine velocity.

Were it not for the interference caused by the yoke supporting the bearings of the cups, the Price meter apparently would record the absolute velocity and, therefore, would over-register with respect to the cosine velocity. Up to angles of about 15° it does this. For angles greater than 15° the indicated velocity is a few per cent. less than the absolute velocity owing probably to the interference of the yoke.

The values presented by Mr. Groat are in good agreement with the average of the Yarnell and Nagler values for right- and left-hand inclinations. This would be expected from the fact that the oscillating movement would give the mean of the right-hand and left-hand effects. Although the Iowa experiments show that the ratio of indicated to cosine velocity depends to some extent on the velocity, this does not appear to have affected Mr. Groat's results appreciably.

Small Price Meter: Vertical Oscillations.—The observations for vertical oscillations are plotted on Fig. 24. Within the range, $\alpha = 0°$ to $25°$, they show an indicated velocity slightly less than the cosine velocity. For angles greater than 25° the indicated velocities are greater than the cosine velocities. This does not agree with the results found by the authors because, in all cases up to the limit of their experiments, the indicated velocities were less than the cosine velocities. Experiments made by the U. S. Geological Survey in 1898,[16] on one of the earlier type Price meters, gave observed velocities in all cases greater than the cosine velocities. On the other hand, the results found by the authors fall between values observed by Brown and Forrest Nagler,[17] M. Am. Soc. C. E.

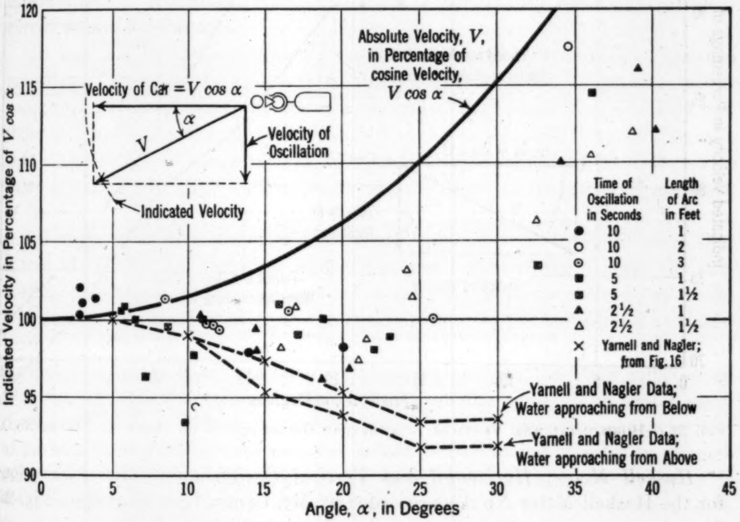


FIG. 24.—VERTICAL TRANSVERSE OSCILLATIONS ON SMALL PRICE METER.

---

[16] Nineteenth Annual Rept., U. S. Geological Survey, Pt. IV, p. 30.

[17] "Stream Gaging," Liddell, First Edition, 1927, p. 144.

The difference may be due partly to the fact that in one case the meters were held at a constant angle of inclination against the current, while in the other they were oscillated, probably at a varying rate. It may be noted that the authors' results on the acoustic Price meter do not show as great departures from the cosine velocity as the electric Price and are in good agreement with Mr. Groat's observations up to an angle of about 20 degrees. Between angles of 20° and 30° the acoustic meter gave indicated velocities about 3% less than the cosine velocity.

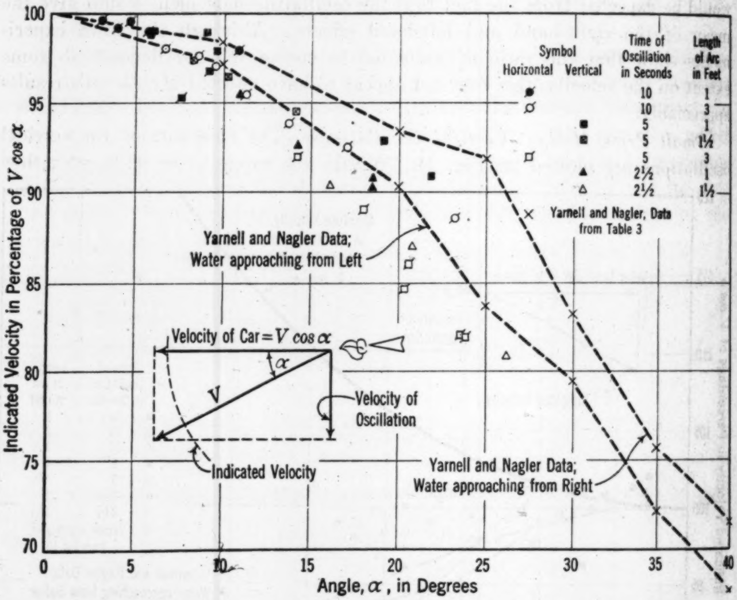


FIG. 25.—HORIZONTAL AND VERTICAL TRANSVERSE OSCILLATIONS ON HASKELL METER NO. 1.

Haskell Meter: Horizontal and Vertical Oscillations.—The observations for the Haskell Meter No. 1 as recorded by Mr. Groat,[18] are plotted on Fig. 25. The observations for both horizontal and vertical oscillations are given, and it can be seen that there is no marked difference between the two, although the latter may have slightly smaller departures. Mr. Groat's results agree well with the departures observed at Iowa for water approaching the meter horizontally from the left-hand side; but they are greater than the departures listed by the authors for water approaching from the right side.

Longitudinal Oscillations: Pulsations.—Mr. Groat's experiments on longitudinal oscillations indicate a slight under-registration, amounting to between 1% and 2%, for both the Price and Haskell meters when the oscillations were not at a high enough rate to give negative velocities relative to the meter. This may have resulted from the rocking motion of the frame which produced a slight vertical component in the velocity of the water relative to the meter. When negative velocities were created, over-registration occurred, as was the case in the authors' experiments, and probably for the same reason, namely, as a result of contacts when the rotor was running backward.

---

[18] *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1246.

The Yarnell and Nagler Experiments on Inclined Meters.—In general, Mr. Groat's observations agree fairly well with those obtained by Messrs. Yarnell and Nagler. The writer is inclined to believe that a part, at least, of the differences between the two, result from the fact that in the authors' experiments a steady flow impinged against a meter held at a constant angle, while in Mr. Groat's experiments the oscillations probably caused both a varying angle and varying velocity. For this reason the latter may be somewhat more representative of actual conditions in turbulent water. The departure resulting from a constant angle of impact is not the same as the average departure resulting from a varying angle having the average value of the constant angle. Furthermore, it is difficult to account for some of the results obtained by the authors.

In the case of the experiments on the meters with horizontally inclined propellers, the writer is at a loss to understand the wide differences in the departures obtained when the water approached the meter from one side and those obtained when the water approached at the same angle from the opposite side. The rotors of these meters are symmetrical and would be expected to show the same effect for a given angle regardless of the direction of approach of the water. The projecting contact box and electric connection on the horizontal supporting arm probably have some influence, but it would seem that it was too far behind the rotor and of too small dimensions to have the pronounced effect indicated in Table 3. For example, note the differences in the departures of the indicated from the cosine velocities for right-hand and left-hand approach. All screw meters listed in Table 3, with the exception of the Medium Ott, gave departures several times greater when the water approached from the left side than when it approached from the right. The writer questions whether these differences are the result of some inherent characteristic of the meters or whether they may not reflect a distribution of velocity or some other condition in the flume when the meter was inclined toward one side different from that existing when it was inclined toward the opposite side. If this was not the case then the screw meter loses an important advantage over the cup meter in that its departure is also dependent on the direction of approach of the water as well as the angle of approach.

The Yarnell and Nagler Experiments on Meters in Turbulent Water.—Taking into consideration that the authors' experiments show that the velocity registered by the meter depends on several factors—including (1) the angle of approach; (2) the direction of approach; (3) the velocity of the water itself; (4) the possibility of negative velocities; as well as (5) the type of meter—it is easy to understand the difficulty encountered in attempting to apply the results of the later experiments to the observations obtained in turbulent water in the first series. While the experiments on inclined meters are of considerable value, they are in effect stream-line ratings of the meter at inclined angles, and do not strictly apply to turbulence consisting of variable velocities and angles of approach. To be of real value they must explain the performance of the meter in turbulent flow.

It is unfortunate that in the turbulence experiments the observations were not taken over a longer period so that the performance of each meter at a given point could be compared. In Experiments Nos. 8 and 9, in which "duplicate readings as shown in Table 1 were taken at each point in the cross-section in order to determine the possibility of error by taking observations for too short a time in turbulent water," it was found that "individual observations of velocity in this turbulent water, taken for the usual length of time, may vary as much as 10 to 20% * * *." Such errors, however, are largely compensating. In Experiments Nos. 8 and 8(*a*) (Table 1) the average difference between duplicate readings at each of the eight points, as well as the maximum difference at any one of the points, is given in Table 7.

TABLE 7.—COMPARISON OF VELOCITIES REGISTERED IN DUPLICATE EXPERIMENTS NOS. 8 AND 8(*a*). (TABLE 1.)

Meter.	Maximum difference at single point, in feet per second.	Average difference for all eight points, in feet per second.
Electric Price	0.28	0.03
Acoustic Price	0.31	0.05
4-blade Hoff	0.46	0.03
Small Ott	0.22	0.12
Large Ott	0.31	0.02

The large maximum differences at single points show that the observations were too short to get the correct average value of the velocity indicated by any of the meters at any one point, and this renders impossible any comparison of the observations of the different meters at a given point. The small average differences, however, indicate the compensating nature of the errors resulting from too short an observation, and this makes the comparison of the average performance at the eight points more reliable.

Although the authors ran Series Nos. 2 and 3 for the purpose of obtaining data that would explain the performance of the meters in turbulent water, this cannot be done by the experiments thus far presented. Not only do the later experiments fail to account for the observations obtained in the turbulent flow experiments, but they indicate either that there is some hitherto unrecognized factor in turbulent flow, or that all the conditions existing in the flume when the flow was turbulent have not been clearly brought out in the paper.

Table 3 and Fig. 15, giving the results with inclined meters, indicate that, in all cases, the 4-blade Hoff meter should register a smaller velocity than the small Ott meter and that as the angle of inclination, α, increases the difference in indicated velocities should increase. Although the magnitude of the departure depends on whether the water approaches the meter from the right or from the left, the direction of approach was presumably the same for both meters, and they should show the same relative departures. Moreover, the authors conclude that if, as was the case in Series No. 1, the electric contacts are made at intervals of several revolutions, all meters will register the correct average of a variable velocity. When, however, the average of the duplicate readings of the small Ott and the 4-blade Hoff meters in Experiments Nos. 8 and 8(a) are compared, it is found that at four of the eight points the Hoff meter gave velocities from 4 to 17% greater than the small Ott meter. In Experiments Nos. 9 and 9(a) the Hoff meter indicated velocities from 12 to 42% greater than the small Ott. This is not explained by insufficient duration of observation or by anything shown by the oblique flow experiments.

The unusual condition of turbulence existing in the flume is shown by the fact that in three out of the eight experiments (two of the total of ten being duplicates) the screw meters registered from 25 to 50% greater velocity than the mean derived from the discharge over the standard weir. Considering that screw meters always under-register oblique velocities, it must be concluded that there was an area of zero or negative flow in the section and that the effective area of forward flowing water was materially less than the total cross-sectional area and therefore resulted in abnormally high velocities. While areas of stagnant or backward flowing water are sometimes encountered in stream-gauging work, particularly immediately behind obstructions, it is generally possible to recognize and to take this condition into account, and such highly abnormal conditions as were apparently created in the authors' flume would be almost unique in field work.

Correction of Observations Made in Turbulent Water.—The writer believes that notwithstanding the impracticability of attempting to obtain the proper correction to current-meter observation by evaluating each of the several factors influencing the registration of the meter, it is possible to carry out field measurements in such a manner as to reduce, if not entirely eliminate, the errors introduced by turbulent flow. There are several methods that should be given consideration by any one who must measure a stream that is more or less turbulent.

Two Measuring Sections.—If two measuring sections are close enough together so that there is no change in the quantity of water passing the sections and yet far enough apart so that the conditions at the sections are different, some indication is given of the amount of turbulence in the stream and also which of the two will give the best result. This is increasingly valuable if the shapes of the sections and the distribution of velocities are materially different. This, of course, does little more than show which of the two sections is the better measuring section. Where, however, measurements must ordinarily be made at an unfavorable section—for example, from a bridge supported by several piers—check measurements made from a boat or from a temporary cable a short distance up stream will give the correction to be applied to measurements made at the lower section.

Elimination of Turbulence by Baffles.—This is obviously the most satisfactory method of dealing with turbulence although it is necessarily confined to use in a stream of small dimensions. If, however, a number of measurements are to be made and accuracy is of more than usual importance, it may be the least expensive of the methods considered.

Measurement of Turbulence.—It seems to the writer that a rather simple device could be constructed that would give the horizontal and vertical directions of the current at each point of observation. Knowing these directions, the correction to be applied to the registered velocity could be made by means of the departures shown on diagrams similar to those presented by the authors. The use of such an instrument would be confined to shallow channels or to streams having a low velocity.

Inclination of Meters.—In the case of rigidly suspended meters it would seem possible to arrange the suspension so that the instrument could be rotated in both horizontal and vertical planes through 15 or 20° from its normal position perpendicular to the section. In this way the direction giving the greatest velocity could be found by trial, and the velocity at the point of observation corrected accordingly. This method could not be used with a cup meter as moderate changes in its horizontal position would not change the velocity indicated. The disadvantage of this method, aside from the trouble of providing the special suspension, lies in the amount of time required at each point. Instead of a single observation of usual duration, four or five observations would be required and these should be for sufficient length of time to get the true average registration and eliminate the effect of differences resulting from pulsating flows.

Use of Two Meters.—If simultaneous measurements are made with two instruments, one of which is a cup meter and the other a screw meter, it may be assumed with considerable reliability that the true discharge lies between the flows indicated by the two meters. Unless the turbulence is caused almost entirely by vertical components of velocity, the cup meter will over-register. The screw meter will always under-register. It remains to determine at what point between the velocities indicated by the two meters the true velocity lies.

That this method merits more than passing attention is shown by the results it gives when applied to the highly abnormal turbulence existing in the authors' flume. Table 2 gives the departures of the velocities indicated by the different meters from the true mean velocity in the flume. Fig. 17 gives the relative registration of the meters when it is assumed that the current approaches the meter from all directions with equal frequency, and lacking any more specific data, it can be assumed that this was the condition in the flume. In general, the curves in this diagram indicate that:

(*a*) Under-registration of the 4-blade Hoff is four times as great as over-registration of Price meter.
(*b*) Under-registration of the medium Ott meter is twice as great as over-registration of Price meter.
(*c*) Under-registration of the small Ott meter is equal to over-registration of Price meter.

Therefore, in order to get a closer approximation to the true velocity than is given by any of the meters alone:

(1) Deduct one-fifth the difference between the Price and the 4-blade Hoff velocities from the Price velocity.
(2) Deduct one-third the difference between the Price and the medium Ott velocities from the Price velocity.
(3) Deduct one-half the difference between the Price and the small Ott velocities from the Price velocity.

The error in the computed velocities using this method is given in Table 8.

It is obvious that for even such highly abnormal turbulence as there was in the flume used for those experiments, and with nothing to indicate the predominating direction of approach, or the nature of the turbulence, this method gives much better results than were obtained by any single meter. Omitting Experiment No. 4, more than one-half the results with the two-meter method are in error by less than 2% and the greatest error is 6.2 per cent. With the single meters more than one-half the results are over 13% in error and the greatest error is 61.6 per cent. In all cases except one, the two-meter method gives a better result than any of the single meters.

TABLE 8.—ERROR IN VELOCITY COMPUTED FROM VELOCITIES REGISTERED BY TWO METERS.

Experiment No.	Small Price and 4-Blade Hoff.			Small Price and Medium Ott.			Small Price and Small Ott.		
	Departure, in percentage.	One-fifth the difference, in percentage.	Error in computed velocity, in percentage.	Departure, in percentage.	One-third the difference, in percentage.	Error in computed velocity, in percentage.	Departure, in percentage.	One-half the difference, in percentage.	Error in computed velocity, in percentage.
3	+18.9* −61.6†			+18.9 −28.3			+18.9 −17.4		
	80.5‡	6.1	+ 2.8	47.2	15.7	+ 3.2	36.3	18.1	+ 0.8
4	−17.4 −20.2			−17.4 −19.9			−17.4 −15.8		
	2.8	0.5	−17.9	2.5	0.8	−18.2	1.6	0.8	−16.6
8	+12.8 −29.5			+12.8 −22.2			+12.8 −12.6		
	42.3	8.5	+ 4.3	35.0	11.7	+ 1.1	25.4	12.7	+ 0.1
8(*a*)	+11.1 −13.5			+11.1 −19.3			+11.1 −12.9		
	24.6	4.9	+ 6.2	30.4	10.1	+ 1.0	24.0	12.0	-- 0.9
9	+ 5.1 − 5.4			+ 5.1 −13.5			+ 5.1 − 3.1		
	10.5	2.1	+ 3.0	18.6	6.2	− 1.1	8.2	4.1	+ 1.0
9(*a*)	+ 9.5 −11.2			+ 9.5 −13.9			+ 9.5 − 1.3		
	20.7	4.1	+ 5.4	23.4	7.8	+ 1.7	10.8	5.4	+ 4.1

* Departure by Price meter, from authors' Table 2.

† Departure by screw meter, from authors' Table 2.

‡ Difference in departures.

In Experiment No. 4 the electric Price meter indicated a lower velocity than either the large or small Ott meters. It under-registered by 17.4%, while the acoustic Price over-registered by 18.5%; there is, therefore, some question concerning this run. Experiments No. 2, 5, 7, and 10 were omitted in the preceding comparison for the reason that in all but one the screw meters indicated from 25 to 50% greater velocity than the true velocity, and in Experiment No. 7 three of the four screw meters over-registered. As has already been pointed out, this indicates a condition that is not entirely explained by the data presented.

Cable Suspension.—The preceding discussion relates to rigidly suspended meters whereas by far the greater number of current-meter measurements are made with cable suspension. Several of the methods described for handling measurements in turbulent water will apply equally well to cable suspension. It should be noted in the case of the two-meter method that the screw meter will not under-register as much with cable suspension as when rigidly suspended owing to the tendency of all meters, including the screw types, to point into the current and so register the absolute velocity.

In view of the results thus far presented the writer would suggest that further work be done to determine completely the effect of direction of approach on the registration of the nearly symmetrical screw meters, and that this be done in a channel of relatively large cross-section to eliminate, in so far as possible, the obstruction caused by the meter itself. It would also seem that turbulent conditions more nearly representative of those actually encountered in practice could be obtained in the 10-ft. hydraulic canal, and that some rational method of correction could be developed by comparing the performance of different meters at the same point with each other and with their registration with oblique flow.

In closing, the writer wishes to commend the work of the authors in their attempt to unravel an intricate problem. He well recognizes the impossibility of foreseeing before embarking on an experimental program, all the various phases that will develop during the course of the work. It is to be hoped that the authors will find the opportunity to carry out further investigation of this difficult question.

Dr.-Ing. Ludwig A. Ott,[19] (by letter).—The solution to numerous problems concerning discharge measurements is eagerly awaited by many engineers. This paper affords a welcome opportunity to discuss one of those problems, namely, the behavior of current meters with runners in turbulent currents.

During the sixteen years since 1914 the writer has repeatedly made practical experiments on this subject and has in general arrived at results similar to those described by the authors. The writer's experience has convinced him, however, that in studying current-meter behavior in oblique currents (always assuming the meter rigidly supported at the measuring section), it is inexpedient to distinguish between "screw wheels" and "cup wheels," the type of runner effecting a difference in grade only but not in principle. The essential feature is not the shape of the runner, but the position of its axis in relation to the plane of the measuring section.

With the shape of any runner fixed, the number of revolutions for a definite flow velocity represents a function of the angle formed by the direction of flow and the runner axis, and in principle all the directions situated on a conic surface around this axis are equivalent. Moreover, only in one position of the axis, that is, perpendicular to the measuring section, is the number of revolutions a function of the angle between the direction of flow and the measuring section, as must be demanded of an instrument for determining the discharge.

---

[19] Math. Mech. Institut, Kempten, Bavaria, Germany.

Whether the runner is better provided with screw blades or cups is a question that cannot be answered offhand as long as behavior in the oblique current is the only consideration. Perhaps some other important factors must be considered, such as the disturbance of the flow, the sensitiveness of recording, the behavior near the surface or the bottom, etc. In either case, it is certain that a cup-wheel should be mounted in the same manner as a screw-wheel, with its axis of rotation vertical to the measuring section. Consequently, the plane of rotation should be parallel to the section.

In oblique currents, cup meters with their axes of rotation set parallel to the profile, show qualities similar to screw meters with their axes placed crosswise, provided the screw-wheel is of such a shape that it will revolve in this position. The writer has made a number of experiments with such runners. They were shaped like five-threaded twist drills and were 1 in. in diameter, 3½ in. long, and had a pitch of 6 in. The generatrix of the helicoidal surface of one of the screws was a straight line, whereas those of the second and third were convex and concave curves, respectively. In the normal position, with the axis perpendicular to the measuring section, all three runners turned with equal numbers of revolutions like any other screw meter of suitable construction. When the axis was placed parallel to the profile the first runner remained at rest. The second and third turned forward and backward, respectively, with an increased speed (because the diameter was small compared with the pitch) analogous to that of right or left-turning cup meters.

To recapitulate: It is not the shape of the runner, but the position of its axis that is essential in researches on current-meter behavior in oblique currents.

In most of his researches the writer has placed the current meters at various angles in flowing water. Presumably Treviranus[20] was the first to adopt this method in his experiments. Experience has proved, however, that sufficiently accurate quantitative evidence is not obtained in all cases. There are errors to be anticipated in oblique currents, because in the equation of current meters:[21]

$$V = A + B\,N \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

the coefficients, A and B, do not, as a rule, change at the same rate for any given angular position of the axis.

Assume that a meter is rated by towing, keeping the axis parallel to the direction of the towing car. (See Fig. 26.) The result is expressed by Equation (1). Then the same meter is rated again with the axis at an angle, α, with the direction of the car (see Fig. 27). The result is then expressed by:

$$V = a + b\,n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

---

[20] Treviranus, "Uber Verbesserungen in der Konstruktion und im Gebrauch des Woltman'schen Fluegels" ("On Improvements in the Construction and Application of the Woltman Current Meter"), *Allgemeine Bauzeitung*, Bd. 26, 1861, pp. 125–164.

[21] With regard to the theoretical derivation of this formula as a simplification of the general equation of the second order, see, Dr. L. A. Ott, "Theorie und Konstantenbestimmung des hydrometrischen Fluegels" ("Theory and Determining the Constants of Current Meters"), Berlin, 1925.

The component coinciding with the meter axis is thus shown to be,

$$v = V \cos \alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

or, substituting the value of V in Equation (2),

$$v = (a + b\, n) \cos \alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

One can see at a glance that Equations (1) and (2) for any angle, α, can only yield identical values for velocities in the direction of the axis when both the ratios, $\frac{B}{b}$ and $\frac{A}{a}$, are equal to the value, $\cos \alpha$. Consequently, in theory this is the fundamental requirement for a so-called component meter.

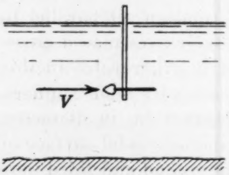


FIG. 26.

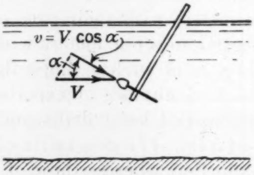


FIG. 27.

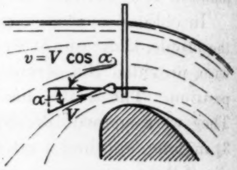


FIG. 28.

In discharge measurements similar to that in Fig. 28, Equation (4) is, as a rule, of little use, because ratings of meters at angular positions of the axis are seldom available and the angle, α, is not known. For these reasons it is desirable that in an oblique current Equation (1) furnish directly the true result, namely,

$$v = A + B\, n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which, v denotes the component of the velocity perpendicular to the measuring section, and n, the number of runner revolutions per second.

For the ideal component meter the ratio, $\frac{n}{N}$, of the number of revolutions in oblique to the number in parallel flow (with equal absolute water velocities) is determined from Equations (1) and (2), as follows:

$$\frac{n}{N} = \cos \alpha - \frac{A}{B\, N} (1 - \cos \alpha) \ldots\ldots\ldots\ldots\ldots (6)$$

An instrument for which ratings by Equations (1) and (2) are available, has an $\frac{n}{N}$-ratio of,

$$\frac{n}{N} = \frac{B}{b} - \frac{a}{b\, N} \left(1 - \frac{A}{a}\right) \ldots\ldots\ldots\ldots\ldots (7)$$

These equations show that the cosine curve for the ratio, $\frac{n}{N}$, is only approximate in revealing the qualities expected from a component meter. They further explain the fact expressed by the authors (Fig. 14), that for different absolute velocities the divergences of the ratio, $\frac{n}{N}$, from the cosine curve are different.

If ratings have been carried out according to Fig. 27, or Equation (2), respectively, it is quite easy to determine the percentage of error which occurs in practice by applying Equation (5) instead of Equation (4). If v denotes the velocity component computed from Equation (5) and Δv, the necessary correction, the result is expressed as follows:

$$\frac{\Delta v}{v} = \frac{b}{B} \cos \alpha - 1 - \frac{A}{v} \left(\frac{b}{B} - \frac{a}{A} \right) \cos \alpha \text{ .... } (8)$$

As an instance the following equations may be quoted which are the results of very extensive researches executed in 1905 with the Ott Meter No. 385[22] (the units are in feet per second):

When $\alpha = 0°$,

$$V = A + B\,N = 0.064 + 1.660\,N \text{ .... } (9)$$

and, when $\alpha = 15°$,

$$V = a + bn = 0.034 + 1.746\,n \text{ .... } (10)$$

Applying Equation (8) when using this meter in an oblique current of 15°, the percentage of correction for the flow component in the direction of the meter axis is,

$$100 \frac{(\Delta v)}{v} = 1.5 - \frac{3.2}{v} \text{ .... } (11)$$

As will be seen, the correction can be either positive or negative. It is + 1%, 0%, and — 1%, respectively, for the velocities, 6.4 ft. per sec., 2.1 ft. per sec., and 1.3 ft. per sec.

The writer has studied particularly the range from 0° to 20° at intervals of 2°, both to the right and to the left. These small intervals were deemed necessary, because otherwise irregularities in the run of the curve may easily be overlooked or assumed to be errors in measuring. Since experience has proved that in practice larger obliquities of current can nearly always be avoided, it appears to be sufficient to limit the range to 20° when determining the cosine qualities of a current meter.

The writer cannot fully agree with the statement in the "Introduction" of the paper restricting the use of current meters to measurements in natural streams and open artificial channels. In Europe, they are the most favored instruments for accurate discharge measurement in large pipe lines. In July, 1929, comparative discharge measurements by various methods (the Allen salt-velocity method, the Gibson method, with Pitot tubes and Ott current meters) were undertaken in one of the penstocks of the Walchenseewerk[23] under the impartial management of the Research Institute for Hydraulic and Hydro-Electric Structures in Munich. The measurements proper were carried out by experienced German and English crews. The results clearly showed the superiority of the current meter for this very kind of gauging, thus increasing every one's confidence in these instruments.

---

[22] Eger, Dix, Seifert, "Die Versuchsanstalt für Wasserbau und Schiffbau, Berlin. Versuche mit Stromflügeln" ("Laboratory for Hydraulics and Shipbuilding in Berlin. Researches on Current Meters"), *Zeitschrift für Bauwesen*, Vol. 57, 1907, p. 266.

[23] Dr. Kirschmer, "Vergleichs-Wassermessungen am Walchenseewerk" ("Comparative Discharge Measurements at the Walchenseewerk"), *Zeitschrift* des Vereins deutscher Ingenieure, Vol. 74, 1930, pp. 521–528.

B. F. Groat,[24] M. Am. Soc. C. E. (by letter).—In connection with this paper it will be of considerable interest and value to discuss a subject that bears directly upon the averaging of current-meter readings taken in a vertical velocity plane, or—what amounts to the same thing—to state those results, giving "rules" similar to those of Cotes, Newton, Simpson, and others, but especially formulated to supply averages in the form of the arithmetical mean. The observations and tables by Messrs. Yarnell and Nagler present opportunities for applying these suggestions.

All hydraulic engineers are familiar with the "two-tenths eight-tenths depth method" of averaging two readings to obtain the mean velocity in any vertical. This rule is based on the hypothesis that the actual vertical velocity curve approximates the arc of a parabola. The rule is exact for a parabola with its axis parallel to the current. It is also exact for a straight-line velocity gradient. It is known that, in particular cases, the actual velocity curve may depart widely from the arc of any parabola. However, if it is possible to state a rule, or rules, based on several point-velocity readings at various depths, such that the arithmetical mean represents the average for a curve of higher order than that of a parabola, the error of the mean can be reduced to an increasing extent as the number of such points is increased. In current-meter work, the appropriate arithmetical mean has the peculiar advantage that the extreme ordinates are not at the ends of the base line (water surface and bottom), as is the case with the usual rules of mensuration. The qualification, "appropriate", is made because there is an infinity of arrangements of ordinates of equal weight.

A general formula for mensuration was rigorously deduced by the writer in a paper, entitled "Mean Value of the Ordinate of the Locus of the Rational Integral Algebraic Function of Degree, n, Expressed as a Weighted Mean of $n + 1$ Ordinates, and the Resulting Rules of Quadrature", read before the American Mathematical Society, at Providence, R. I., on September 12, 1930.[24a] It furnishes not only the well-known formulas of mensuration (Newton's rule, Simpson's rule, etc.), as special cases, but also supplies, as other special cases, the rules conforming to the specification for current-meter readings (as previously stated) at various depths in the vertical-velocity plane.

These results may be stated briefly under eight headings, as follows:

(1) The "two-tenths eight-tenths depth method" is misnamed;
(2) Range of validity of mensuration rules;
(3) Weight ratios for "two-tenths eight-tenths depth method";
(4) Arithmetical mean of three ordinates;
(5) Arithmetical mean of four ordinates;
(6) Arithmetical mean of five ordinates;
(7) Arithmetical mean of six ordinates; and
(8) Arithmetical mean of ten ordinates.

(1).—The "Two-Tenths, Eight-Tenths Depth Method."—This method is misnamed. While the error caused by this inexactness may be moderate in some cases, yet such a name should be discarded because it is misleading.

---

[24] Cons. Engr., Brookline, Mass.

[24a] *The Am. Mathematical Monthly*, Vol. XXXVIII, No. 4, April, 1931, p. 212.

The correct relative depths (relative distances from the ends of the base line in mensuration) for the two velocity readings, when measured from either the water surface, or the bottom, are exactly, and, respectively, $\frac{1}{2} \mp \sqrt{\frac{1}{12}}$, or, approximately, in decimals, 0.211325 and 0.788675. It should be observed that this not only represents the true area of a parabola through the two points, but also the area bounded by a straight line passing through the two points, the base line, and end ordinates (water surface and bottom).

(*2*).—*Range of Validity of Mensuration Rules.*—The foregoing relation is a general truth in mensuration; if n points lie on the locus of a rational, integral, algebraical equation of lower degree than $n - 1$, then the true formula for n points will not only be a true rule for the locus of a rational, integral, algebraical equation of the $n - 1$th degree, but, also, for all loci of equations of lower degree than $n - 1$. Therefore, the rule for a parabola works on a straight line; the rule for a cubic works on a parabola and on a straight line; the rule for a quartic works on a cubic, a quadratic, and a straight line, etc.

(*3*) *Weight Ratios for "Two-Tenths Eight-Tenths Depth Method."*—The rule for the so-called "two-tenths, eight-tenths depth method" results by making the weight of the middle (mid-depth) ordinate of the parabola equal to zero by properly locating the other two ordinates, so that Cotes' rule (the basis of Simpson's rule), 1:4:1, is replaced by 1:0:1, thus giving equal weights to two ordinates. It was shown in the paper before the American Mathematical Society that the ordinates may be located anywhere and their respective weights calculated; or *vice versa*, the weights may be chosen and the ordinate locations calculated. This method is applied to curves of higher order, thus obtaining further simple rules for averaging ordinates (point-velocity readings in a vertical velocity curve), some of which are indicated in Table 9.

Under Heading No. 7, in Columns (4) and (5), $c = z - z^2$. Thus, r is a function of the coefficients of a cubic. The rules tabulated determine an average ordinate in terms of the arithmetical mean of n properly located ordinates.

In Table 9 the column headings, x (x'), y (y'), and z (z'), refer to the relative values of the abscissas corresponding to ordinates symmetrically located in pairs with respect to the center of the base line. Thus, $x + x' = 1$, etc. When there is an odd number of ordinates, z and z' (Column (4)) become identical as the relative abscissa of the mid-ordinate.

Abscissas are relative distances from either end of the given base line (surface, or bottom, for current-meter velocities). When a great number of averages, or resulting areas, are to be determined from drawings, or profiles, a diagram of radiating lines may be drawn on a separate piece of tracing cloth, so as to touch the ends of a base line within the limits of the tracing cloth and also to intersect the base line at intermediate points relatively distant from the ends by amounts proportional to the abscissas concomitant with the rule to be used. The diagram with its base parallel to the given base, and with its end lines touching the ends of the given base, will locate immediately all the intermediate points, along the given base line, at which ordinates are to be erected. The arithmetical mean of these n ordinates is the average value which, when multiplied by the length of the given base, gives the area enclosed by the profile, the base line, and two ordinates erected at the two ends of the given base on the drawing. The two end lines on the diagram may be used to locate convenient consecutive ends for a succession of bases into which a long line is to be divided.

TABLE 9.—SPECIAL QUADRATURE RULES FOR THE MEAN VALUE OF A FUNCTION, DERIVED FROM THE GENERAL RULE.

Heading No. (1)	x (x'). (2)	y (y'). (3)	z (z'). (4)	r. (5)	Ordinate spacing. (6)	Remarks. (7)
(4) ARITHMETICAL MEAN OF THREE ORDINATES ($n = 3$)						
4	$\frac{1}{2} \mp r$	………	$\frac{1}{2}$	$\sqrt{\frac{1}{8}}$	r	………
	0.1464466	………	$\frac{1}{2}$	………	………	………
	0.8535534	………	………	………	………	………
(5) ARITHMETICAL MEAN OF FOUR ORDINATES ($n = 4$)						
5	$\frac{1}{2} \mp r$	$\frac{1}{2} \mp \frac{1}{3} r$	………	$\sqrt{0.15}$	$\frac{2}{3} r$	………
	0.1127017	0.3709006	………	………	………	………
	0.8872983	0.6290994	………	………	………	………
(6) ARITHMETICAL MEAN OF FIVE ORDINATES ($n = 5$)						
6	$\frac{1}{2} \mp r_1$	$\frac{1}{2} \mp r_2$	$\frac{1}{2}$	$\sqrt{\frac{1}{4} - \frac{7 \mp \sqrt{11}}{48}}$	$r_1 - r_2 : r_2$	Ordinate spacing not uniform.
	0.0837513	0.3127293	$\frac{1}{2}$	0.416248743	………	r_1 in Column (5).
	0.9162487	0.6872707	………	0.187270705	………	r_2 in Column (5).
(7) ARITHMETICAL MEAN OF SIX ORDINATES ($n = 6$)						
7	$\frac{1}{2} \mp 5r$	$\frac{1}{2} \mp 3r$	$\frac{1}{2} \mp r$	$\sqrt{\frac{1}{4} - c}$	$2r$	………
	0.07920706	0.2475242	0.4158414	………	………	………
	0.9207929	0.7524758	0.5841586	0.08415859	………	………

These consecutive bases need not necessarily be equal, but had better be chosen so as to secure greater accuracy, depending upon the nature of the profile to be examined. The end ordinates do not enter the arithmetical mean. If they did, the result would not be so accurate, since the relative weights of end ordinates, for symmetry about the center, are less than unity. The rules give the exact area under the curve of a rational, integral, algebraic curve of degree, $n - 1$ (that is, of a degree less by 1 than the concomitant number of ordinates stated). If the ordinates are drawn to any curve of more or less smooth shape, the arithmetical mean of the ordinates is an approximation to the mean value of the ordinate of that curve, the degree of approximation increasing with the number of ordinates (and with the degree), as previously explained. The rule for n ordinates not only gives the exact area for degree, $n-1$, but also for all lower degrees.

Universal Applicability of the Rules.—There can be no doubt that proper use of Rules (4) to (8), inclusive, will lead to greater accuracy in reducing field work, whether in water measurement or other classes of engineering. The accuracy of the formulas can be checked by applying them to curves for which the proper ordinates have been calculated. Use the radicals, r, because the symmetry causes them to disappear.

Limitation on the Use of Weddle's Rule.—When the base is divided into six equal parts by seven ordinates, one at each point of division, including the ends of the base, Weddle's rule states that the weights are, respectively, 1, 5, 1, 6, 1, 5, and 1, but it is not exact for seven ordinates to a curve of the sixth degree. The correct weights for this case are, respectively, 41, 216, 27, 272, 27, 216, and 41; the sum equals 840.

The arithmetical mean of ten ordinates representing generally the area under the locus of an equation of the ninth degree would furnish a most facile method for practical use, since it requires nothing more than addition and shift of a decimal point. The weights for ten ordinates which divide the base into nine equal parts are, beginning at either end, respectively, 2 857; 15 741; 1 080; 19 344; 5 778; 5 778; 19 344; 1 080; 15 741; and 2 857; total, 89 600.

(8).—Arithmetical Mean of Ten Ordinates.—Until the more exact rule for the arithmetical mean of ten ordinates is adduced, the arithmetical convenience of ten ordinates, together with an improvement in accuracy, can be had by using the rule for five ordinates on a base divided into two halves, just as one applies Simpson's (Cotes') rule to two, or more, sub-bases. The relative locations of the ordinates for this purpose are: 0.04187563; 0.15636465; 0.2500000; 0.34363535; 0.45812437; 0.54187563; 0.65636465; 0.75000000; 0.84363535; and 0.95812437. The arithmetical mean of ten ordinates thus located is the desired average to about seven places of figures; that is, accurate for two arcs of quintics which, together, pass through all ten points.

Tables 7 and 8 present the essence, and show the advantages, of the method of "aggregates" or the "statistical method".[25] In any good gauging section, the ratio of discharge to the aggregate revolutions per unit time (or aggregate velocities, if preferred) for readings taken at given points is a determinate function of the elevation of the water surface. When the discharge is constant, as in a mill-race, the function is constant. The usual statement that accuracy cannot be made less than 2% is too conservative. If one takes care to eliminate unknown influences as far as possible, including such conditions as rapidly fluctuating stages, and then determines proper constants by means of proper aggregates, the discharge can be gauged within a fraction of 1 per cent. Most of the difficulties with current meters have arisen when trying to harmonize varying readings when the aggregate used was altogether too thin. Engineers must distinguish carefully between accidental errors and systematic, or constant errors.

---

[25] "Characteristics of Cup and Screw Current Meters; Performance of These Meters in Tail-Races and Large Mountain Streams; Statistical Synthesis of Discharge Curves," *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), pp. 819, 838.

Figs. 23, 24 and 25 show that there is some kind of relation between such widely separated conditions of rating as those at Iowa and in the rating tank of the University of Michigan. It does not seem that there should be any very close agreement between ratings in turbulent water and ratings in steadily flowing water, even if the average angle of approach and average direction should be the same in the two cases. This is best illustrated by the differences visible in Fig. 24. However, it is interesting to know that the relation is closer than might be suspected.

FORREST NAGLER,[26] M. AM. SOC. C. E. (by letter).—Exhaustive investigation on the effect of turbulence on current meters has long been lacking, and the paper under discussion fills a very definite gap in water-measurement practice. The work is exceptionally valuable, not only because of the methods used, but also because of the wide variety of current-meter forms tested. The meter performances investigated have to deal with solid angles, although they must be analyzed in horizontal and vertical planes for the purpose of showing their characteristics in diagrammatic form. It seems, however, that a polar diagram is slightly more appropriate than the rectangular diagram used by the authors for showing the detail meter characteristics.

Figs. 29 to 34 are submitted to show a line of investigation carried out by the writer, that parallels with surprising closeness a small portion of the work covered in the paper. Figs. 29 to 32, inclusive, show the detail work done on some of the meters available.

The initial experiments were made on a large sized Price meter in excellent condition. The curves in Fig. 29 show how it is affected by flows at angles with the horizontal. On account of the obvious inaccuracy of the meter in question, the work was not carried out farther than is indicated in the diagram. Later, in order to obtain an approximate comparison of the large Price meter with other meters, an arithmetical average was made between the vectors of Curve *D*, Fig. 29, and Curve *B*, Fig. 30, the latter being made on a small-sized, cup type of meter. The varying effect of different forms of frame will undoubtedly influence the form of Curve *B*, which is made in a horizontal direction, but for approximate purposes it was considered to be sufficiently accurate. The large Price meter was the least accurate of all those examined, and was particularly inaccurate if there was a preponderance of flow coming at an angle with the horizontal. It registered a velocity in excess of the desired resolved components under all conditions.

The small Price meter (Fig. 30) shows up unusually well up to angles of 25°, but it will be noted that this applies only for conditions of angular flow evenly distributed, horizontally and vertically. For such a condition this meter is probably very accurate, but it ceases to be so the moment there is a preponderance of angularity either in the horizontal or the vertical direction.

---

[26] With A. O. Smith Corporation, Milwaukee, Wis.

For example, if the preponderance is in a horizontal direction, the meter registers an excess velocity; and, if it is vertical, it registers less than the true amount up to about 40 degrees.

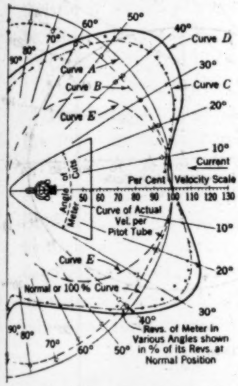


FIG. 29.—TESTS WITH A LARGE PRICE METER.

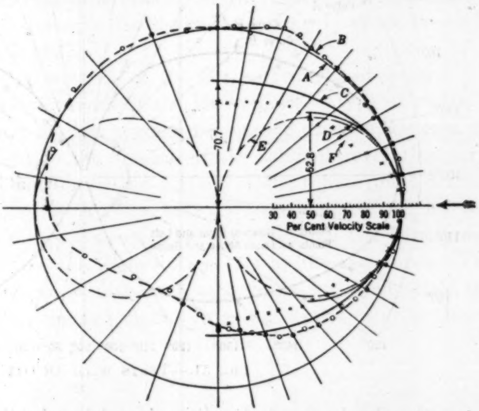


FIG. 30.—TESTS WITH A SMALL PRICE METER.

Examination of the results obtained with the screw type of meter shows that it is the ideal form for use in conditions of disturbed flow, because it makes the same effort to give resolved components regardless of the direction of the angularity. Of the two meters examined (see Figs. 31 and 32), the Ott meter is considerably the better. Incidentally, it may be mentioned that this applies not only to performance, but to the mechanical structure of meter bearings and housing. On the basis of the correction to be applied to Curve 4 in Figs. 33 and 34, making its semi-minor axis 35 instead of 32, it becomes evident that these two meters show results not differing widely, but in the same direction, from a true value.

The various meters were mounted on a vertical or horizontal axis on a frame installed in a 42-in. pipe in which water at a temperature of between 90 and 100° Fahr. flowed to a depth of about 14 in., at a velocity of approximately 4 ft. per sec. This velocity was very constant, the discharge being from several large condenser pumps. The pipe was of concrete, uniform in section, fairly smooth inside, and several hundred feet long, with constant slope. The flow was even and practically parallel by reason of the regular section, high velocity, and long smooth channel. Pitot tube readings were taken simultaneously with meter reading both at the zero position and at the angular positions so as to show whether the movement of the meter affected the flow. The tube was used for this purpose rather than to check the actual velocity reading of the meter, although this latter comparison is available.

In plotting results of the tests the curves could not be fixed with sufficient accuracy by inspection, so the method of least squares was adopted for determining the probable location and form of characteristic curve. This was used for all meters, except the large Price, which was handled entirely by inspection and by an arithmetical average of the vectors of the two curves.

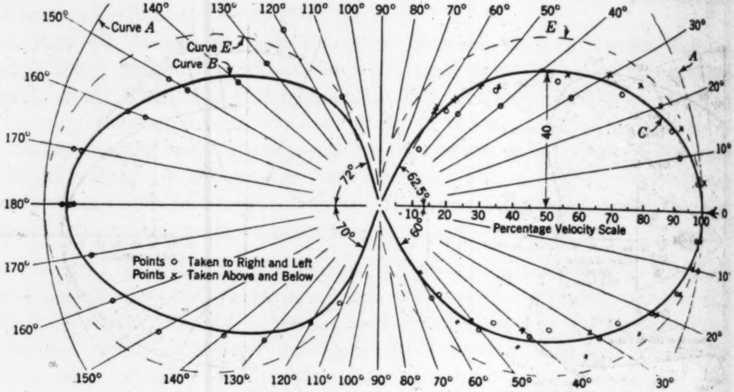


FIG. 31.—TESTS WITH AN OTT METER.

On the basis of the foregoing determination, the following semi-minor axes were determined:

Small Price .... $B = 70.7$
Ott .... $B = 40$
Haskell .... $B = 32$

The determination for the small Price meter was also made on the basis of Curves D and $2D$, shown in Figs. 30 and 33, in order to make possible a direct comparison of the probable characteristics of the three meters. The various ellipses determined on the foregoing basis are shown in Figs. 29 to 32, in which the circles, A and E, are the two basic curves. To afford better comparison these various curves are replotted together in Fig. 33. Bringing together all curves in this form suggests a method of correcting differences between any two meters. In order to facilitate this comparison, the separate curves may be defined as follows:

Curve 1 represents the performance of a large Price meter rigidly supported and subjected to angular flow uniformly distributed. (If the preponderance is known or suspected to be in either the vertical or horizontal direction, reference, respectively, to Curve D (Fig. 29), and Curve E (Fig. 30), will indicate the direction and possible magnitude of error.)

Curve 5 represents a 100% curve, which would be expected from any good make of meter so supported as to be free to turn and measure the thread of the current, provided the support be such that the revolving element is given no sidewise motion.

Curve $2C$ represents the performance of the small Price meter rigidly supported and subjected to angular flow uniformly distributed. (The note given under Curve 1 applies here, referring to the horizontal and vertical curves (B and F) of Fig. 30.)

Curve 2*D* represents a forced mathematical interpretation of the performance of a small Price meter made for the purpose of comparison only, and should not be used hereafter.

Curve 0 shows the ideal or cosine curve. If the meter had this characteristic it would always register only the desired component; hence it would be ideal (inertia effects neglected).

Curve 3 represents the performance of the Ott, screw type of meter.

Curve 4 represents the performance of the Haskell, screw type of meter.

Referring to Figs. 33 and 34, the average percentage excess registered by the Price meter is equivalent to the volume of the solid of revolution (*A*), divided by the area of that portion of the surface of the sphere the intercept of which is Curve 0, enclosed by a cone with an apex angle of 40 degrees. Similarly, the average percentage deficiency registered by the Ott is obtained by dividing the volume of the solid of revolution (*B*) by the same area. The determination of these averages would serve as a basis for reconciling the discrepant results obtained by the two meters in actual work. Similar determinations for the various other meters can readily be made.

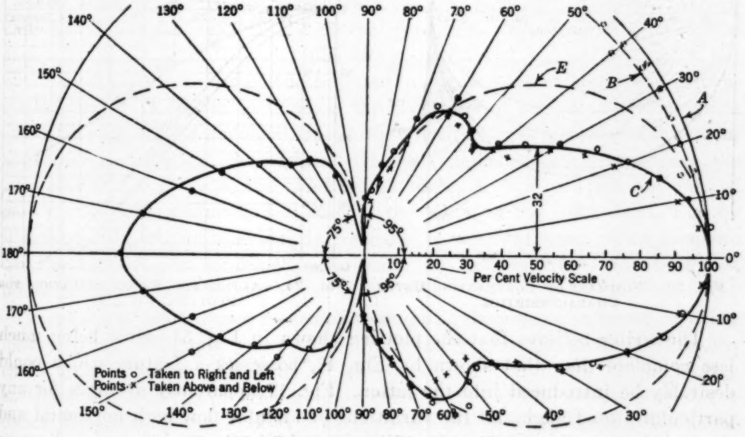


FIG. 32.—TESTS WITH A HASKELL METER.

In forming the computations, integration is difficult by reason of the complicated forms of some of the surfaces, but a simpler method is available. Fig. 34 is plotted directly from Fig. 33, the radii vectors of the various curves being plotted as percentages of the corresponding radii vectors of the zero circle for angles up to 45 degrees. These curves are plotted according to the small percentage scale at the left of Fig. 34. On this diagram the areas, *A* and *B*, and, consequently, their respective volumes of revolution about the vertical axis, correspond to the similar areas and volumes for Fig. 33. Furthermore, the area of the surface described previously is now determined as the area of a circle of radius (20) in Fig. 34.

By inspection of Fig. 34 it is seen that the various curves are approximately parabolic in form, particularly near the origin. Therefore, the various volumes may be determined by multiplying the areas of the base of the solid of revolutions by one-half its height. Dividing this volume by the area of base, in turn, leaves a mean ordinate of one-half the height of the actual ordinate of the curve. This permits of a simple solution by dividing the vertical percentage scale by 2, resulting in the large percentage scale indicated at the left as plus and minus.

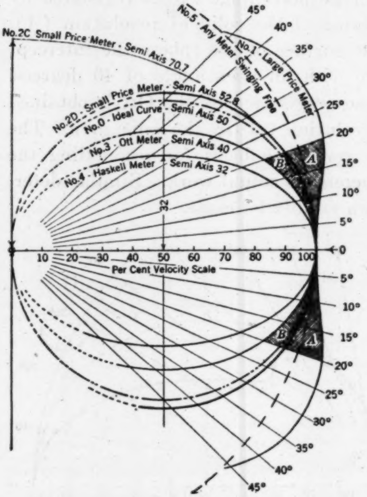

No.2C Small Price Meter - Semi Axis 70.7
No.5 - Any Meter Swinging Free
No.1 - Large Price Meter
No.2D - Small Price Meter - Semi Axis 52.8
No.0 - Ideal Curve - Semi Axis 50
No.3 - Ott Meter Semi Axis 40
No.4 - Haskell Meter Semi Axis 32
32
A
B
45°
40°
35°
30°
25°
20°
15°
10°
5°
0
10
20
30
40
50
60
70
80
90
100
Per Cent Velocity Scale

FIG. 33.—SUMMARY OF CONTRASTING METER CHARACTERISTICS.

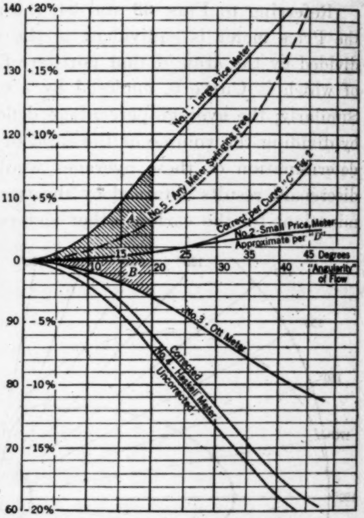


FIG. 34.—ALTERNATE RECOMMENDATION FOR PLOTTING FIG. 33.

The writer believes that the plotting shown in Fig. 34, while being much less complete than that shown by Fig. 17, possesses a feature which could desirably be introduced into the latter. Fig. 17 apparently averages for any particular stated angle, all the variations of angular flow both horizontal and vertical, but at that angle only. Fig. 34 is basically an integrated diagram and the plotted points of the curves at any angles (say, 20°) are not an average of angular effects at 20°; they are a summation or average of all the effects from zero to 20°, assuming that these effects exist a uniform length of time for all degrees up to that maximum. It is an attempt to define turbulence as being represented by constantly varying impact angles within and up to a certain maximum. This appeals to the writer as being a truer picture of turbulent flow and its angularity effect on a current-meter head.

The writer's work, done in 1913 and 1914, was premised on the assumption that all discrepancies in current-meter observations could be traced to disturbed conditions in the flow, and that all types of meters would register alike according to their still-water ratings if the flow were of stream-line characteristics, straight, uniform, and parallel to the direction in which the meter is pointed. The extent to which the flow departs from this condition fixes the amount of divergence of any meter from true indication, and, of even greater importance, the extent of the disturbance fixes the relative discrepancy between the indications of any particular meters.

Analysis of the curves in Figs. 29 to 34 led quite positively to many of the conclusions arrived at in the paper under discussion. It led further to the conclusion that the only style of meter that may be used with absolute certainty, is a rigidly supported instrument with identical characteristics for horizontal, vertical, and all intermediate angularities of flow. This means that its axis of rotation must coincide with the direction in which it is pointed; in other words, it must be a screw type of current meter. The work led further to an attempt to build screw meters with modified helical surfaces generated by curved lines, rather than by straight-line elements, these curved lines being so disposed as to present the concave side of the helical surface to the normal direction of flow, so that any side angularities would amplify angular effects to counteract the evident under-registering indication of the existing types, and give more nearly the desired cosine function.

This work attained only a moderate success, sufficient, however, to indicate that it was basically in the correct direction. The writer believes that such a meter can be built very much along the lines of a small Ott meter, but with a more nearly correct cosine response to angular flow. It has been demonstrated that it is simple to increase the semi-axis of the small Ott diagram shown in Fig. 31, to such an extent that positive readings are obtained at angularities of 70 and 80 degrees. It was further demonstrated that the adherence of all points to the ideal cosine curve, *E*, is a question not only of curved generating elements, but also of the shape of the head itself, Fig. 32 illustrating a peculiarity evidently traceable to shape of head. The complete development of this combination presents an excellent field for improved present current-meter types.

Fig. 34 is presented as a means of reconciling discrepancies found in using meters of all different types. The discrepancy between meters of known characteristics is an indication of the degree of turbulence or angularity encountered. What is of greater importance, it might also offer a clue to the correct flow even if that flow is different from the flow indicated by either of the meters. In such application it is desirable to have the two meters under comparison, of diverse characteristics, so that correct flow will lie between them. This is merely because it takes quite a stretch of the imagination to use two such meters as the Ott and the Haskell (which, at an indicated angularity of 30 in Fig. 34, may under-register, respectively, 6% and 11½%), and increase the discharges of both of them by the amount in question merely because they happen to differ by about 5½ per cent. Such a procedure, however, is in line with a logical analysis of the data provided in the paper and in this discussion.

An investigation of completely shrouded meters would be very interesting. The shrouded Fteley meter analyzed[27] by Charles P. Rumpf, M. Am. Soc. C. E., shows an approach to the true cosine circle (Curve *A*), indicated by a semiminor axis of about 45 units. Similar analysis of the large shrouded screw meter made by Gurley would be of interest.

---

[27] *Engineering News*, Vol. 71, No. 20, May 14, 1914, p. 1083.

To the writer the significant showing of the paper under discussion is in the extreme over-registering possible with both screw and cup type of meters under turbulent conditions. The under-registering of the Haskell type under good, but angular, flow conditions is the sole contrast, but it should be pointed out that under-registering resulting in a turbine efficiency exceeding 100% (as has actually happened) is instantly discarded; whereas, a result of 80% caused by over-registering carries no red warning flag labeled "impossible".

The authors are contributing a most excellent means of judging relative meter performance, to the end that wide inaccuracies in either direction may be prevented.

ROBERT E. HORTON,[28] M. AM. SOC. C. E. (by letter).—The paper under discussion presents the results of three different lines of investigation of the relation of the velocity registered by current meters of different types to the velocity of the stream. It suffers somewhat from seeming, if not actual, contradictions of statement and from lack of clarity. It is also difficult to check some of the figures and conclusions owing to absence of the original tabular data of the experiments.

To avoid uncertainty or confusion it has been found necessary at times in this discussion to set forth briefly what the writer understands to be the meaning of some of the statements and results presented by the authors.

Current meters for measuring the velocity of water are of two general types. In the cup type, illustrated by the Price meter, the axis of the rotor (as distinguished from the axis of the meter), is normal to the line of flow. Such meters are actuated by the excess of dynamic pressure on a flat surface over that on a curved or conical surface having an equal projected area. In the screw or helical type, the axis is parallel to the line of flow. Such a meter is actuated by impact of water on the inclined surfaces of the vanes.

Current meters serve, among many others, for three important uses:

1.—Determination of points on stage-discharge relation curves or rating curves for open-section stream-gauging stations. For this purpose cable suspensions are generally used.

2.—Measurement of flow in artificial channels, such as head-races and tail-races in power plant tests. Such channels often contain artificial turbulence, which is sometimes violent.

3.—Measurement of flow in channels, such as irrigation ditches, etc., commonly with smooth flow and slight turbulence. For these conditions rod mountings are frequently used. In the experiments described by the authors, rod mountings alone were utilized and, as was pointed out in the paper, the results and conclusions cannot with certainty be applied, unmodified, to meters used with cable suspension.

The most important use is in the measurement of natural streams. For this purpose the current-meter method is often the only one economically available. It permits the measurement of flow without loss of head and without disturbing natural conditions. Turbulence is nearly always present, particularly at higher stages. It seems worth while, therefore, to give some attention to the question of turbulence in natural streams.

---

[28] Cons. Hydr. Engr., Voorheesville, N. Y.

The Nature of Turbulence.—In this discussion turbulence will be considered as consisting of eddies. For this purpose, an eddy may be defined broadly as a rotating mass of water regardless of the distribution of velocity within it, provided all the particles rotate around a common axis. A vortex is an eddy with a special velocity distribution such that for all radii, R, of rotating filaments the product, Rc, is constant, in which, c, denotes the tangential velocity of a filament at the radius, R. A vortex represents a stable system of motion in the sense that it can be destroyed only by friction. A part of the fluid may also rotate as a solid, that is, with uniform angular velocity at all radii. As the result of friction there may be a combination of a vortex and rotation *en masse* as, for example, a condition precisely intermediate. Table 10 indicates the characteristics of three types of rotating fluid masses.

TABLE 10.—CHARACTERISTICS OF ROTATING FLUID MASSES

Type No.	Name	Relation of c to R	Relation of angular velocity, ω, to R	Relation of speed of rotation, N, to R
1	Free vortex..............	$c = \frac{\text{constant}}{R}$	$\omega \propto \frac{1}{R^2}$	$N = \frac{c}{2\pi R^2}$
2	Rotation *en masse*......	$c = 2\pi N R$	$\omega = \text{constant}$	$N = \text{constant}$
3	Intermediate condition.	$c = \text{constant}$	$\omega \propto \frac{1}{R}$	$N \propto \frac{1}{R}$

The origin of eddies, particularly vortices in fluids, has been described by Dr. L. Prandtl, taking into account recent researches.[29] This need not be repeated further than to note that eddies originate where there is an obstruction to flow, such as a projection from the boundary of a natural channel, sufficient to produce a surface of discontinuity in the adjacent fluid. If there is a sufficiently marked difference in direction or magnitude of velocity on the two sides of this surface, an eddy is formed which grows in magnitude until it breaks away, carrying a mass of fluid with it. A new surface of discontinuity is then formed down stream from the projection and the operation is repeated.

The application of these results to the circulation of water in a natural stream will next be considered.

In a natural stream, eddies originate (except as hereafter noted) wholly around the margin of the cross-section and tend to travel normal to the boundary of the channel at the point of origin of the eddy, or at right angles to the line of flow. The normal component of eddy motion, is, however, combined with the velocity of the stream so that the resultant direction of motion is diagonally down stream and toward the center or surface of the stream. As the eddy travels, its energy is dissipated as heat. In a very wide channel, eddies thrown off horizontally at the sides may be dissipated before reaching the center of the channel. Eddies thrown off at the bottom, if weak, or if the channel is very deep, may not reach the surface. Strong eddies reaching the surface are spread out as visible "boils".

---

[29] "The Physics of Solids and Fluids—with Recent Developments," by P. P. Ewald, Th. Pöschl, and L. Prandtl, authorized translation by Dougall and Deans, pub. by Blackie & Sons, Ltd., Lond. and Glasgow, 1930.

A vortex (like a smoke ring) carries with it part of the fluid out of which it is originally formed. Since eddies originate around the perimeter of the cross-section and hence in regions of low velocity and low unit volume energy, their propagation through the cross-section tends to equalize the velocity, as is in fact observed.[30] This theory of the circulation of water in rivers as influenced by turbulence has other important implications not relevant to the present discussion. One, however, may inquire: What becomes of the kinetic energy released by the equalization of velocities resulting from eddy cross-currents? On the other hand, the retardation of velocity due to viscous shear tends to equalize the velocities in a vortex and reduce that vortex to a simple case of rotation *en masse,* and any intermediate condition between that of a free vortex and that of simple rotation *en masse* may obviously occur. Changes taking place in eddy turbulence at different distances from the boundary where the eddies originate, have been studied experimentally.[31]

Mutual interference and coalescence of eddies do not greatly change the situation. They tend in many cases to accelerate the dissipation of energy and the disappearance of eddies. In other cases they accentuate the disturbance and may produce abrupt and violent changes or reversals of velocity such as are sometimes observed in current-meter work in violently turbulent water. Another interesting and important fact which, however, the writer has not seen described in engineering literature, is the "calving" of eddies of the vortex type. Energy changes due to external friction may cause the breaking up of an original vortex eddy formed at the boundary of the channel after it has risen part way or entirely to the surface. Under suitable conditions the original vortex eddy disappears, but it is succeeded by a series of small, active vortices, each containing a portion of the original vortex fluid. This phenomenon of "calving" of vortices may be successfully demonstrated by a simple experiment if carefully performed.[32] A minute drop of ink on the end of a toothpick is gently shaken into a glass of perfectly still water. A black vortex ring travels downward, expanding and presently breaking up into a group (usually five or six) of small, perfect vortex rings, radiating outward and downward. These, in turn, break up into other groups, the entire series resembling successive star clusters from a rocket. The breaking up of original eddies, therefore, is not followed necessarily by a heterogeneous or hit-and-miss system of cross-currents, but, in part, by a new order of active eddies. The writer believes that a theory of the behavior of current meters in turbulent flow predicated on the composition of rotary and longitudinal motion (assuming the rotary motion to be of the character described), will provide a reasonably accurate "model".

---

[30] "The Physics of Solids and Fluids—with Recent Developments," by P. P. Ewald, Th. Pöschl, and L. Prandtl, 1930, p. 281.

[31] "Caratteri della Turbolenza nelle Condotte Cilindriche," *L'Elettrotecnica*, No. 15, 1930, pp. 730–733.

[32] "The Splash of a Drop," by A. M. Worthington, Smithsonian Inst., Annual Rept., 1894.

Effect of Artificial Turbulence.—The first part of the paper gives the results of a series of experiments wherein artificial turbulence was induced in the experimental canal by horizontal and by vertical paddles (see Fig. 11). In these experiments the mean velocity in the canal was 2.0 ft. per sec. Different types of meters were used, and with each meter a velocity determination was made at each of the eight points in the cross-section as indicated on Fig. 10. Tests were made first with each meter, using horizontal paddles, and these were duplicated. Tests otherwise identical with the preceding were then made with vertical paddles, which were also duplicated. The resulting indicated velocities at the index points are shown in Table 1. The means of these velocities are given in Table 11.

TABLE 11.—MEANS OF VELOCITIES AT EIGHT POINTS OF MEASUREMENT IN FLUME WITH PADDLES

(Units are in feet per second)

Meter (1)	Horizontal paddles (Test 8) (2)	Vertical paddles (Test 9) (3)
Small Price, electric	2.18	2.12
" " acoustic	2.36	2.28
Hoff	1.74	1.90
Small Ott	1.76	1.95
Medium Ott	1.63	1.75
Large Ott	1.77	1.73
Mean	1.91	1.96

The figures in Columns (2) and (3), Table 11, for a given type of paddle, show the results obtained by taking one-half the sum of the means of the velocities measured at the eight index points in each of the two tests.

It was generally found that the agreement between mean velocities at a given index point in the two tests with a given meter and a given type of paddle was excellent. Therefore, only the average of the two tests is included in Table 11.

Comparing the velocities obtained, respectively, with horizontal and with vertical paddles, it will be seen that the differences are small but significant in the case of most of the meters, but that they greatly exceed the allowable limits for accurate measurement in some cases, particularly for the Hoff and small Ott meters. If the average of the six determinations of the mean velocity in the channel for the six different meters is taken, the result with horizontal paddles is 1.91 ft. per sec., while with vertical paddles it is 1.96 ft. per sec. In view of the close agreement of these figures and their small departure from the true mean velocity of 2 ft. per sec., the writer believes that the differences in velocity shown by individual meters in this test with the two types of paddles are probably due mainly to fixed differences in distribution of velocity rather than to variations in the total amount or character of turbulence in the channel resulting from the types of paddles used.

There is evidence in Table 1 of a decidedly abnormal distribution of velocities in the channel. Furthermore, it appears probable that the turbulence produced by the paddles differed from that existing in a natural stream channel so materially as to make the applicability of these results in practice somewhat doubtful. This is unfortunate since these are the only experiments included in the paper in which turbulence occurs to any important extent in the water in which the meters were suspended.

Two conclusions, however, may be drawn from these experiments:

1.—In very turbulent water the small Price meter may over-register from 6 to 18%, while the Hoff and Ott meters may under-register by amounts varying with the type of meter from 2½ to 18 per cent. By methods somewhat similar to those used by the authors, Kirschmer and Esteren have confirmed[33] the over-registration of the Price meter and the under-registration of the Ott meter.

The Price meter over-registered more with horizontal than with vertical paddles, while the Hoff and Ott meters generally under-registered. This probably resulted from the difference of behavior of these meters in fixed cross-currents, as revealed by the third series of experiments.

2.—A second conclusion is that by the use of two or more meters so selected that over-registration of one kind is balanced by under-registration of the other kind, comparatively accurate results can be obtained with almost any degree of disturbance of the current by baffles or other obstructions. The use of two meters, one of which is known to over-register and the other to under-register, is a practice which has been followed by the writer and others in measurements of very turbulent flow in power plant tests and other similar situations. The authors' experiment so far as it goes, confirms the utility of this practice.

Energy Consumed in Friction as a Measure of Turbulence.—Correction factors to be applied to different types of current meters in use in turbulent water are greatly to be desired. In fact, the determination of such factors seems to be the proper primary object of experimental investigations such as those by the authors. It is to be regretted that they did not go much farther along this line. It might be inferred that since there is no quantitative method of measuring turbulence, such correction factors, even if available, would be of little practical utility. Although the point seems to have been overlooked heretofore, the writer believes that an index of the amount of turbulence in a stream can readily be obtained from the ordinary slope formula. Take, for example, the Manning formula,

$$v = \frac{1.486}{n} R^{\frac{2}{3}} \sqrt{s} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

---

[33] *Zeitschrift* des Vereines Deutscher Ingenieure, v. 74, pp. 1499–1504; also, *Engineering Abstracts,* Inst. C. E., Lond., No. 46, January, 1931, pp. 20–21.

in which, v is the mean velocity of the stream, in feet per second; n is Kutter's coefficient of roughness; R, the hydraulic radius; and s, the slope or fall, in feet per foot. By transposition this becomes,

$$s = \frac{v^2\, n^2}{(1.486)^2\, R^{\frac{4}{3}}} \quad \text{.......................} (13)$$

The slope, s, is the height through which each unit of volume or mass of the flowing water must fall per foot of linear travel in order to provide the energy necessary to overcome friction. If it is assumed that the energy consumed in overcoming friction represents turbulent or eddy energy which is converted into heat, then it is evident that Equation (13) is a direct measure of the rate at which eddy energy is being transformed. On the other hand, if the slope in the reach under consideration is uniform, then the rate at which energy is being dissipated or transformed into heat is necessarily identical with that at which eddy energy is being produced in the channel. In other words, it is a measure of the rate at which energy is being impounded, so to speak, in eddy formation around the channel perimeter.

Since the velocity under the conditions assumed is known, with a known rate of inflow, and the hydraulic radius is also known, it follows that a function—such as Equation (13)—which apparently is directly proportional to the amount of energy of turbulence in the stream, can always be derived for any cross-section if the coefficient of roughness, n, is known. This coefficient can easily be determined, of course, from slope measurements in connection with the known rate of discharge.

This method of procedure should show a definite relation between the function indicated by Equation (13) and the error of mean velocity determined by any given type of meter at a given velocity in the stream. It is possible that the character of turbulence may differ in two different streams even when the eddy energy per unit volume is the same. Whether the effect of the resultant turbulence on the operation of current meters would or would not be the same in the two cases is a question which cannot be answered definitely except as the result of experiments.

Suggested Program of Experiments.—The possibility of developing an index of turbulence, the value of which could readily be determined in connection with current-meter measurements and applied to their correction, deserves most careful consideration. Such a correction, if established on a sound basis, is as necessary and as well justified as, for example, the correction of astronomical observations for refraction. In spite of the extensive, although exceedingly unsatisfactory, literature regarding current-meter operation in turbulent water, the writer believes that much more could be accomplished along this line, by carrying out a campaign of experiments with the following technique and procedure.

With an experimental channel such as that used by the authors, for example, deliver an accurately known quantity of water into the channel in such a manner that it shall flow as smoothly and as nearly free from initial turbulence as possible. Next, induce artificial turbulence in the water by means of rotors similar, for example, to the propeller of an electric fan. These rotors could easily be arranged on the bottom and sides of the channel at points up stream from the measuring section so as to produce cross-currents simulating those produced naturally by eddies thrown off from the channel perimeter in the manner already described.

The rotors should be propelled by electric motors or otherwise in such a manner that their speed can be controlled and determined so that the amount of energy put into the water in producing eddies or turbulence can be measured accurately. Measurements of the velocity would then be made at a sufficiently large number of index points in the cross-section to give an accurate determination of the apparent velocity with each type of meter, with different known velocities in the channel, and with different known amounts of energy being expended in producing turbulence.

If it develops that the departure of the measured mean velocity from the true mean velocity by any given type of meter bears a definite functional relation to the two controlled variables—the mean velocity, and the amount of energy expended in producing turbulence—a start will have been made toward a quantitative solution of the problem.

In that event it would seem worth while to conduct experiments under field conditions in natural river channels. In the case of the natural river channel the procedure would necessarily involve measuring the flow at a cross-section down stream, for example, from a dam or reservoir, where the quantity of inflow to the reach containing the measuring section could be accurately controlled and determined, and where the coefficient of roughness, *n*, for the adjacent reach of the river could also be determined.

Turbulence in air is measured quantitatively and its characteristics recorded graphically by means of a Dines pressure-tube anemometer. In experiments such as those suggested herein it would be a valuable adjunct to the results to have turbulence graphs taken by some type of Pitot-tube recorder. Possibly such an instrument might be attached directly to the current meter for experimental work. It is evident that either a means must be found for expressing turbulence quantitatively in terms of known variables and applying corrections to velocities measured by various types of current meters in turbulent water, or a considerable degree of uncertainty must be attached to measurements of stream velocity by current meters where the flow is markedly turbulent. The writer hopes that experiments along the lines indicated may be made forthwith.

Varying Velocity Effect.—The authors very properly subdivide the effect of turbulence on the operation of a current meter into two categories:

1.—The effect of varying velocities in the line of flow. This may briefly be called the "varying velocity effect". The expression, "line of flow", will here be used to indicate the direction of the current with no turbulence. It is also parallel with the direction of the axis of the meter as ordinarily used.

2.—The effect of currents approaching the meter at an angle to its axis, or at an angle to the normal line of flow, a condition which occurs in turbulent flow. This may be called broadly the "cross-current effect".

The second investigation by the authors consisted in determining the relation between the reading with the meter moved alternately backward and forward in the channel, and the reading of the same instrument held stationary. In these experiments the meter was suspended at the center of the cross-section, and velocities of 1, 2, 3, and 5 ft. per sec. were created in the channel by the operation of the gates. No baffles were used.

Again, referring to Fig. 12, it appears that each meter was swung back and forth parallel with the line of flow, through a range of 2 ft., by a wheel and crank, using a connecting rod 2 ft. in length. The abscissas are in terms of the ratio of the velocity of the crank-pin of the rotating wheel to the stream velocity.

The maximum rate at which the meter travels in swinging back and forth was a little greater than, but may be assumed for purposes of discussion to have been practically indentical with, the velocity of the crank-pin. The velocity of the meter relative to the water varied approximately from $v - c$ to $v + c$ and *vice versa,* in which, v is the stream velocity and c, the crank-pin velocity. The conditions, therefore, were closely similar to those which would exist in a stream containing eddies if the eddies passed the meter in succession in such a manner that the tangential component of the velocity of a filament of the eddy of a radius equal to the radius of the crank-pin was added to the stream velocity and if this was succeeded shortly by a second eddy, the opposite or retrograde-moving side of which passed through the center of the meter.

Following out the idea that the energy involved in turbulence is a controlling factor in the errors of registration of current meters in turbulent water, the following analysis has been made, assuming that the meter passes, successively, through the advancing and retrograding sides of eddies, v being the stream velocity in the line of flow, and c, the maximum tangential eddy velocity encountered by the meter.

Let it be assumed that the conditions are such that the actual velocity encountered by the meter while passing through a reach of flowing water of length, L, increases uniformly from $v - c$ to $v + c$, the velocity at any moment being,

$$v' = (v - c) + \frac{2\,c}{L}\,l \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

in which, l is the distance the meter has traversed from the initial point. The kinetic energy per unit volume of water encountered by the meter at any point is,

$$E' = \frac{w\,v'^2}{2\,g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

The mean kinetic energy per unit volume of water encountered in traversing the length, L, is,

$$E_{\text{ave.}} = \frac{w}{2\,g\,L}\int_{v-c}^{v+c} v'^2\,d\,l \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

but $d\,l = \frac{L}{2\,c}\,d\,v'$. Substituting and integrating, Equation (16) becomes:

$$E_{\text{ave.}} = \frac{w}{4\,g\,c} \times \frac{v'^3}{3} = \frac{w}{2\,g} \times \frac{1}{6\,c}\,[(v + c)^3 - (v - c)^3] \ldots\ldots (17)$$

Expanding and simplifying,

$$E_{\text{ave.}} = \frac{w}{2\,g}\left[v^2 + \frac{c^2}{3}\right] \ldots\ldots\ldots\ldots\ldots\ldots(18)$$

Let $c = k\,v$; then,

$$E_{\text{ave.}} = \frac{w}{2\,g}\left(v^2 + \frac{k^2\,v^2}{3}\right) = \frac{w}{2\,g}\left(1 + \frac{k^2}{3}\right)v^2 \ldots\ldots\ldots(19)$$

The average kinetic energy per unit volume at the constant velocity, v, is,

$$E_v = \frac{w\,v^2}{2\,g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(20)$$

If, therefore, the velocity varies between the limits described in the distance, L, the meter will encounter excess kinetic energy in each cubic foot of water over that encountered with uniform velocity amounting to,

$$E_{\text{ave.}} - E_v = \frac{k^2}{3} \times \frac{w\,v^2}{2\,g} \ldots\ldots\ldots\ldots\ldots\ldots(21)$$

It appears, therefore, that with a velocity varying uniformly there is available an amount of kinetic energy to re-act on the meter greater than with equal mean but uniform velocity. The effect of the varying velocity, therefore, should be proportional to $\frac{k^2}{3}$, and should be equal to $\frac{k^2}{3}$ times a factor, f, which may or may not be a simple proportionality factor. Fig. 12 shows the percentage of over-registration for two types of small Price meter with varying velocities and a mean velocity of 2.0 ft. per sec. In the experiments the velocity did not vary uniformly from $v - c$ to $v + c$. This point will be considered later.

In Table 12, Column (1) shows the value of the crank-pin velocity ratio, $\frac{c}{v}$; Column (2), the kinetic energy excess over that for a uniform velocity, or the value of $\frac{k^2}{3}$, expressed as a percentage. Column (3) shows, for the small Price meter, the percentage excess of meter reading over that for uniform velocity; Column (4) shows the computed excess, using a value of the factor, $f = 0.60$; and Columns (5) and (6) give similar data for the U. S. Geological Survey improved Price meter. Observed excess meter readings in Columns (3) and (5) are taken directly from Fig. 12. The values of f are derived by taking the average ratio of these quantities to the energy excess given in Column (2).

Fig. 35 offers a comparison with Fig. 12 for the two small Price meters with symbols showing the computed over-registration of these meters as given in Table 12 for various values of c. The agreement of the computed with the observed values is practically perfect and is certainly within the limits of errors of observation except for the smaller values of k, for which, apparently, the friction or inertia of the meter rotor has a stabilizing effect and reduces the over-registration. Fig. 12 shows that, for the Mensing-Ott and the three-blade meters, the medium Ott meter, and the Hoff four-blade meter, the value of f is zero. For the small Ott and the Hoff three-blade meters, the value of f is about 0.033.

TABLE 12.—RELATION OF OBSERVED OVER-REGISTRATION OF PRICE METER IN A CURRENT OF VARYING VELOCITY, TO OVER-REGISTRATION COMPUTED FROM ENERGY EXCESS

$k = \frac{c}{v}$	Energy excess ratio, $\frac{E_{\text{ave.}} - E_v}{E_v} \times 100$	EXCESS METER READING, PERCENTAGES			
		Small Price		U. S. Geological Survey Improved Price	
		Observed	Computed, $f = 0.60$	Observed	Computed, $f = 0.54$
(1)	(2)	(3)	(4)	(5)	(6)
0.5	8.3	1.5	4.8	1.0	4.0
1.0	33.3	16.0	20.0	14.0	18.0
1.5	75.0	45.0	45.0	42.0	40.5
2.0	133.0	80.0	80.0	76.0	72.0
2.5	208.3	125.0	125.0	116.0	112.0
3.0	300.0	175.0	180.0	160.0	162.0

An interesting fact brought out by the preceding analysis is that the over-registration of a cup meter in varying velocity can be determined analytically and expressed rationally in terms of the excess kinetic energy of varying flow.

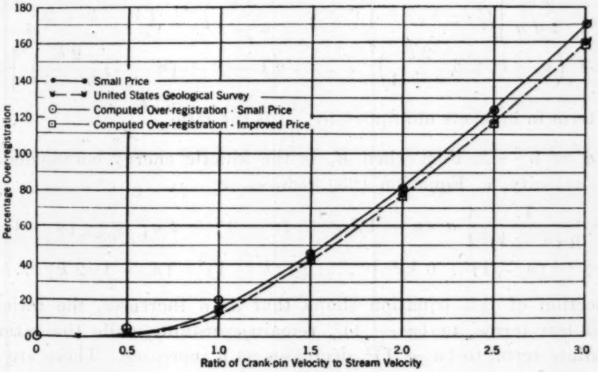


FIG. 35.—RELATION OF OBSERVED ERROR OF PRICE METERS IN VARYING FLOW TO ERROR COMPUTED FROM EXCESS KINETIC ENERGY.

The data indicate that *f* is a constant for a given meter and also that it is an important characteristic of the type of meter. It is apparent that the value of this constant can readily be determined for a given type of meter by the use of apparatus similar to that described by the authors. Some such standard apparatus should be developed and determinations of the factor, *f*, should be made in conjunction with the rating of meters which are to be used in turbulent water.

The authors' experiments were confined to a single current velocity, $v = 2$ ft. per sec., with various crank-pin velocities. It now appears that the results are applicable to any current velocity, providing the correction factor used is that applying to the proper value of $k = \frac{c}{v}$. In other words, the correction for over-registration of a Price meter, due to varying velocity, is the same for $v = 3$ and $c = 1.5$ as for $v = 2.0$ and $c = 1.0$, since $\frac{c}{v} = 0.50$ in both cases. Experimental verification of this point, however, is desirable. The preceding analysis is based on uniform velocity variation from $v - c$ to $v + c$.

Referring to Fig. 36, *a b* is the velocity curve for uniformly varying velocity. It is evident that the velocity curve may follow an infinity of paths—one of which is shown by *m n*, Fig. 36—in changing from $v - c$ to $v + c$, for each of which the mean velocity will be v, the minimum, $v - c$, and the maximum, $v + c$.

If the velocity increases from $v - c$ to $v + c$ by n equal steps,

$$E_{\text{ave.}} = \frac{w}{2\,g\,n}\left[(v - c)^2 + \left(v - c + \frac{2\,c}{n-1}\right)^2 + \left(v - c + 2\,\frac{2\,c}{n-1}\right)^2 + \left(v - c + 3\,\frac{2\,c}{n-1}\right)^2 + \ldots \left(v - c + (n-1)\,\frac{2\,c}{n-1}\right)^2\right] \ldots\ldots (22)$$

or, since $c = k\,v$,

$$E_{\text{ave.}} = \frac{w\,v^2}{2\,g\,n}\left[(1 - k)^2 + \left(1 - k + \frac{2\,k}{n-1}\right)^2 + \left(1 - k + 2\,\frac{2\,k}{n-1}\right)^2 + \left(1 - k + 3\,\frac{2\,k}{n-1}\right)^2 + \ldots \left(1 - k + (n-1)\,\frac{2\,k}{n-1}\right)^2\right] \ldots (23)$$

The term in brackets multiplied by $\frac{1}{n}$, is the ratio, $\frac{E_{\text{ave.}}}{E_v}$.

Let $a = 1 - k$, then when E_v is the kinetic energy corresponding to a uniform velocity, v, Equation (23) reduces to,

$$\frac{E_{\text{ave.}}}{E_v} = \frac{1}{n\,(n-1)^2}\Big\{ a^2\,(n-1)^2 + [a\,(n-1) + 2\,k]^2 + [a\,(n-1) + 4\,k]^2 + [a\,(n-1) + 6\,k]^2 + \ldots [a\,(n-1) + (n-1)\,2\,k]^2 \ldots\ldots\ldots (24)$$

Inspection of this equation shows that as n increases, the ratios of the first and last terms, to $(n - 1)^2$, remain constant, while the ratio of any intermediate term, to $(n - 1)^2$, decreases as n increases. There are n terms. Consequently, as n approaches infinity the average value of the terms will decrease. The minimum value will be that corresponding to a uniform change of velocity, or the ratio of the excess kinetic energy to that for a constant velocity will approach asymptotically the value of this ratio for a uniformly changing velocity. The same result can be readily obtained in a particular case by numerical examples, using varying values of the number of steps, n, by which the velocity changes from $v - c$ to $v + c$. The general result may be stated in terms of the following theorem:

> In a mass of water having a mean velocity, v, and in which the velocities of different filaments vary between the values $v - c$ and $v + c$, the kinetic energy per unit volume is a minimum when the velocity changes uniformly from $v - c$ to $v + c$.

If the change of velocity is not uniform, the excess kinetic energy will be $> \frac{1}{3} \times \frac{c^2}{v^2} \times \frac{w\,v^2}{2\,g}$, and the over-registration of the meter will be greater than that for the same v and c with a uniform velocity gradient. In the authors' experiments the velocity variation followed approximately the law,

$$v' = v - c \cos \frac{l}{L}\,\pi \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

Hence, the excess energy was greater than $\frac{1}{3} \times \frac{c^2}{v^2} \times \frac{w\,v^2}{2\,g}$.

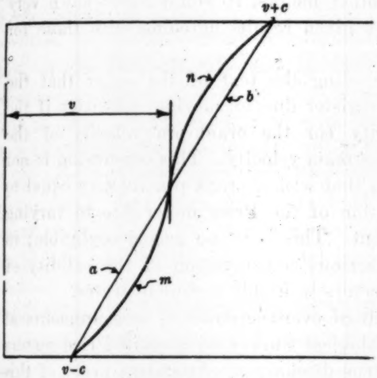


FIG. 36

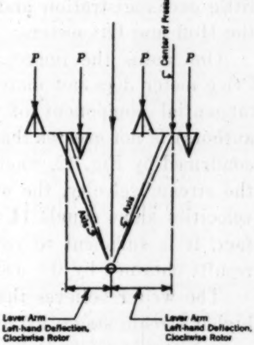


FIG. 37

For the cosine velocity curve the total kinetic energy per unit volume is,

$$E = \frac{w}{2\,g} \int_0^L \left(v^2 - 2\,c\,v \cos \pi\,\frac{l}{L} + c^2 \cos^2 \pi\,\frac{l}{L} \right) d\,l \ldots\ldots (26)$$

Integrating and reducing,

$$E_{\text{ave.}} = \frac{w}{2\,g} \left[v^2 + \frac{c^2}{2} \right] = \frac{w}{2\,g} \left[1 + \frac{k^2}{2} \right] v^2 \ldots\ldots\ldots\ldots (27)$$

The excess kinetic energy in this case is $\frac{k^2}{2} \times \frac{w\,v^2}{2\,g}$, as compared with the value, $\frac{k^2}{3} \times \frac{w\,v^2}{2\,g}$, in case of uniform velocity variation.

Where the experiments are made with an apparatus that gives a cosine velocity variation as in the authors' experiments, the factor, f, should be computed on that basis. The values of f for two Price meters on the two different bases are as follows:

	Uniform change	Cosine law
Small Price....................................	0.60	0.40
U. S. Geological Survey Improved Price....	0.54	0.36

It is perhaps immaterial what law of velocity change is used, provided a fixed standard is adopted for experimental purposes. Uniform change of velocity has a quality of simplicity and definiteness and gives the minimum excess energy per unit volume.

Referring to Fig. 12, the measurements taken by the Haskell meters are not comparable with those taken with the other meters for which results are shown on this diagram. The reader needs to be cautioned not to fall into the error of assuming that the Haskell meter, if used in the same manner as the other meters, would over-register—as the diagram indicates—to substantially the same extent as the Price meter. The contrary is the fact as shown at the end of the section entitled "Effect of Fluctuations in Speed of Flowing Water". Had the Haskell meter been operated to register every fifth or tenth revolution, as was the case with the other meters, it would have shown very little over-registration and would have given results agreeing with those for the Hoff and Ott meters.

One gains the impression from reading the text of the paper that the Price meter does not materially over-register due to varying velocities if the tangential component of eddy velocity (or the crank-pin velocity of the authors) is not greater than the mean stream velocity. This conclusion is not confirmed by Fig. 12, where it appears that with a crank-pin velocity equal to the stream velocity, the over-registration of the Price meter due to varying velocities alone equals 14 to 16 per cent. This is by no means negligible; in fact, it is sufficient to require most serious consideration of the validity of results obtained by the use of Price meters in highly turbulent water.

The writer believes that as a result of over-registration, measurements at higher stream stages by the U. S. Geological Survey with small Price meters are often materially in excess of the true discharges. Abundant proof of this can be adduced, but need not be presented here. To some extent this effect of over-registration is partly offset by the fact that most of the stream measurements at higher stages are made when the river stage is falling or after the crest of a flood has passed; the slope and, consequently, the velocity of the stream are, therefore, somewhat less than the normal slope and velocity for the same stream stage.

In view of the preceding, the writer cannot agree with the authors' conclusions that the meters tested will not be influenced by varying velocity of flow if up-stream currents are not encountered. Certainly, an error of 14 to 16%, as shown on Fig. 12 for the Price meter, with no up-stream velocity, does not confirm this conclusion.

Current-Angle Effect.—Meters of the cup type measure the effect of the current in the direction of its actual flow, whether or not they are held parallel to the line of flow. When operating in a cross-current such a meter with its axis parallel to the line of flow will measure approximately the actual velocity of the cross-current. If it measures correctly the velocity of the cross-current in the direction in which that current is flowing, then the error of the measurement (as distinguished from the error of the meter), will be the difference between the measured velocity and the product of the measured velocity by the cosine of the angle of the cross-current to the line of flow.

If, under these conditions, the meter does not correctly measure the actual velocity of the cross-current in its direction of flow, then the inherent error of the meter will be the difference between the actual velocity of the cross-current and the indicated velocity. If, as in the third series of experiments, the angle of the cross-current is known, its cosine value can be determined and the inherent error of the instrument can be found, as was done by the authors by comparing the velocity indicated by the meter with the true cosine value of the cross-current velocity. Meters of the cup type fail to some extent, and meters of the screw type to a greater extent, to measure the true velocity of the cross-current.

In practice, both varying velocity and cross-current effects are nearly always present together and the percentage error of the measured velocity is reflected by the product of the two percentage ratios of indicated to true velocities. If, due to varying velocity alone, the meter indicates a velocity of 115% of the true velocity and if, due to cross-current effect, the meter indicates a velocity of 95% of the cosine velocity, then for this particular angle of deflection the velocity as actually measured is 1.15 × 0.95, or 1.0925 times as great as the true velocity in the line of flow.

The authors found that the Price meter in particular showed a greater current-angle effect when the meter head was turned to the left than when turned to the right. A similar phenomenon was observed[34] by Charles P. Rumpf, M. Am. Soc. C. E., and also by E. H. Brown and Forrest Nagler,[35] Members, Am. Soc. C. E. These observations apparently establish the reality of the phenomenon. In the writer's experience, Price meters generally have clockwise rotation, but they are made for both clockwise and counter-clockwise rotation. The authors do not state in which direction the rotor revolved. Neither do they state the point at which, or the manner in which, the angle of deflection was measured. Presumably, it was measured by setting the meter at a certain angle before immersion.

Two possible causes of asymmetrical current-angle effects with Price meters are: (1) Deflection of the meter to a different angle than the measured angle, due to differential torsion of the suspension rod; and (2) a similar effect due to unbalanced dynamic pressure on the revolving rotor, something like that resulting from twirling a baseball in throwing a curve.

The first effect might result from the excess pressure on the cups over that on the cones or backs of the cups, as shown on Fig. 37, with the result that the lever arm of the torque on the suspension rod is greater when the meter is deflected in one direction than when deflected in the opposite direction, depending on whether the rotor has clockwise or counter-clockwise rotation. From the viewpoint of a scale model, the speed of a meter rotor and the velocity of a current in a fluid of the density of water, may be of the same order as the speed of rotation and velocity of a baseball in air.

Assuming that the meter is sufficiently rigid against torsion, the combined effect of these causes, if any, on the meter rotor, could readily be determined by experiments with a clockwise rotor with equal right and left-hand deflections of the meter, followed by identical experiments with a counter-clockwise rotor.

---

[34] *Engineering News*, Vol. 17, 1914, pp. 1083–1084.

[35] *Proceedings*, Eng. Soc. of Western Pennsylvania, Vol. 30, 1914–15.

In this connection, it seems worth while to suggest that something might be added to the knowledge gained through experiments, such as those of the authors, on current-meter operation by studying the form and behavior of the water filaments passing the meter rotor.

This result could easily be accomplished with slight depths of submergence by suspending crystals of some soluble coloring matter, such as potassium permanganate, at intervals across the stream just up stream from the meter. This would provide colored streaks in the fluid acting on the meter without in any way interfering with the meter operation.

RICHARD VAN VLIET,[36] JUN. AM. SOC. C. E. (by letter).—During the thirty years, 1900 to 1930, numerous experiments have been performed by various parties in an endeavor to ascertain the errors resulting from the use of a current meter when subjected to various conditions of flow. A review of the results obtained indicates that the several parties practically agree on the errors caused by a meter when subjected to the various manipulated flows. The results obtained by experiments, such as those performed by Messrs. Yarnell and Nagler, are not very helpful to one who desires to correct actual field measurements made in perturbed waters.

A study of the results of these several experiments leads one to believe that measurements of perturbed water, made by a current meter, are subject to considerable error. The writer believes that the error in measurement due to turbulent water has caused more consternation among the members of the Engineering Profession than is warranted. The best proof of this is the result obtained where simultaneous readings measured by a current meter are compared with those measured by a carefully calibrated weir, or sluice, or by volumetric means, etc. Volumetric measurements are extremely valuable and are far more applicable for correcting actual stream measurements of perturbed waters than experiments performed in laboratories with manipulated flows.

In contrast to the voluminous data obtained as the result of laboratory experiments with manipulated flows, there is a dearth of information available as to the relation that exists between discharges measured in the field by current meters and those measured by carefully calibrated weirs, sluices, etc. Probably the best series of experiments representing the comparison between the discharge measured by a current meter and that obtained from calibrated sluices, were those performed at the Assuan Dam on the Nile by Sir Murdoch MacDonald and H. E. Hurst.[37] The sluices at the Assuan Dam had previously been calibrated carefully by volumetric means so that the discharge through them could be computed with negligible error.

Forty experiments were performed, using either a small or a "medium" sized Price meter. On inquiry, the type of instrument referred to as the "medium" sized meter was found to be that corresponding to the commonly known large sized Price meter with a 6-in. bucket wheel. Of these forty experiments, twenty-five were made with the small Price meter. Numerous velocity determinations were made for each vertical. As the sluices were carefully calibrated, a comparison between the discharges measured by the current meters and those given by the sluices will be a fairly good indication of the error encountered in measuring a stream by a Price meter. Table 13 is a summary of the results obtained by the small Price meter.

---

[36] Asst. Engr., with Robert E. Horton, Cons. Hydr. Engr., Voorheesville, N. Y.

[37] "Measurement of the Discharge of the Nile Through the Sluices of Assuan Dam," *Minutes of Proceedings*, Inst. C. E., Vol. CCXII, 1920–21, Pt. II, pp. 228–301.

TABLE 13.—COMPARISON OF DISCHARGES AS MEASURED BY CURRENT METER AND BY SLUICE

Limit of discharge, in cubic meters per second	Number of tests	Average percentage of error*
Less than 700	2	+ 0.4
700 to 800	8	+ 3.5
800 to 1 000	4	+ 0.5
1 000 to 1 200	5	− 0.1
1 200 to 1 488	6	+ 1.7
Total	25	......

* Minus, when meter discharge is greater than sluice discharge.

The maximum under-registration for any individual test was 7.0%, with a discharge of 703 cu. m. per sec., whereas the maximum over-registration was 5.6%, with a discharge of 1 174 cu. m. per sec. The measurements made by current meter were performed in the natural stream channel in the vicinity immediately below the dam.

As in all work of this type some results are obtained, which deviate far from the average. It is only by performing a sufficient number of such experiments that an estimate of the average error can be reached. The question of correcting current-meter measurements of turbulent water does not permit the use of results obtained by small-scale models or manipulated canal currents. Data of the type similar to those obtained in connection with the experiments at the Assuan Dam are meager, and the writer feels that it is only by similar experiments that the mooted question of correcting current-meter measurements can be settled.

It is unfortunate that the authors have included only papers of investigations conducted in America in their bibliography (see Appendix). A paper by Mr. M. A. Hogan gives the results of numerous experiments performed in Great Britain.[38] In general, the various results agree with those obtained by Messrs. Yarnell and Nagler. The general conclusions reached as a result of Mr. Hogan's studies and experiments were, as follows:

1.—The error in registration was dependent on the position of the meter with reference to the sides of the channel.

2.—The error in registration caused by the obliquity was greater in a small channel than in a large channel.

3.—The results showed a smaller error in registration for a shielded meter, such as the Stoppani, than for an Ott or an Amsler meter.

4.—Where considerable turbulence exists, a meter of the Price type will give results in excess of the true value. On the other hand, a meter of the screw type will give results which are deficient as compared with the true value.

---

[38] "River Gauging," by M. A. Hogan, Dept. of Scientific and Industrial Research, Lond., H. M. Stationery Office, 1925.

The experiments relating to oblique flows were performed on Amsler, Ott, and Stoppani meters only, because the writer thought that the experiments performed by Brown and Nagler[39] on the Price meter were sufficient to establish the peculiarities relating to this type and did not warrant any duplication.

DAVID L. YARNELL AND FLOYD A. NAGLER, MEMBERS, AM. SOC. C. E. (by letter).—The many valuable suggestions for additional experimental work on current meters presented in the discussions reveal the unsettled status of the current meter, in the minds of many engineers, for water measurement under certain conditions. Although unable to proceed with this additional experimentation at this time, it is hoped that the work may be undertaken and reported upon in the near future. The specific suggestions which seem to warrant further investigation are:

(*a*) Comparative tests of flow measurements by the various meters with cable suspension in turbulent water.
(*b*) Further investigation of the cause for the difference in registration of the various meters in oblique flow approaching the meter from the right or left.
(*c*) Experiments on propeller shapes to develop a meter with the true cosine characteristic of rotation through a wide angle.
(*d*) Experiments with meters to determine the correction for turbulence along the line suggested by Mr. Leach.
(*e*) Experiments on meter registration with varying turbulence, following the suggestion of Mr. Horton concerning the rotating paddle; and,
(*f*) Comparative tests of the behavior of various meters near the water surface and sides and bottom of the channel.

It is with gratification that the writers have noted the comparisons by Messrs. Leach and Forrest Nagler, showing that their data checked exceptionally well with the results of similar tests performed by other experimenters.

The conditions of turbulence produced by means of paddles in the initial series of tests were probably more excessive than would ever be encountered in practice, as was surmised by Messrs. Nettleton, Leach, and Horton. Extreme conditions of turbulence were produced for the very purpose of exaggerating the discrepancies in registration of the various meters. It is a fact that the meters encountered sharp changes in speed and direction of flow. Eight points of measurement in a section under such violent disturbance were scarcely sufficient to establish a valid comparison between the average velocity recorded by the meters in comparison with the weir. However, in spite of this fact, it may be observed that with only one exception the meters listed in Table 2 arranged themselves in the same general order of accuracy in over-registration or under-registration as that indicated by the curves in Fig. 17, when the integration of the average deviation is computed in the manner suggested by Mr. Forrest Nagler.

In connection with these same tests, the impression that reliable rating curves were not available should be corrected. Recent curves were at hand for every meter tested, and values from these curves were used in the compilation of Table 2, which, therefore, should be quite accurate. The unusual turbulence is responsible for the variations in the listed values in Tables 1 and 2, since the ordinary period of observation (40 to 90 sec.) was insufficient to secure a true integration of the flow at each point (as suggested by Mr. Bennett and mentioned by the writers under "Constancy of Experimental Conditions"). Furthermore, the various meters had rotating elements of different diameters, and, in turbulent water, the large propeller might be actuated by a water filament of large diameter which had an average speed entirely different from the velocity of the smaller filament that strikes the meter with the small propeller, although the center of both propellers may have been at exactly the same point in the stream.

---

[39] *Proceedings*, Eng. Soc. of Western Pennsylvania, Vol. 30, 1914–15, pp. 280–323.

In performing their elementary experiments on meters with rod suspension, the writers purposely eliminated the possible variations that might have resulted when the meter was allowed the freedom of cable suspension. The performance of a meter with cable suspension would vary somewhat with different tail-pieces, and varying lengths of cable. With cable suspension, it is believed that the various meters would arrange themselves in practically the same order with regard to over- or under-registration, and as concluded by Mr. Leach:

> "The screw meter will not under-register as much with cable suspension as when rigidly suspended, owing to the tendency of all meters, including the screw types, to point into the current and so register the absolute velocity."

Some experimenters have found that the Price meter rotates only slightly slower with cable suspension than when rigidly held on a rod; hence, experiments with cable suspension would probably favor the screw meter more than the writers' experiments with rigid rod suspension have indicated.

Mr. Horton suggests some causes for the difference in registration of the Price meter when the current strikes at equal angles from the right or left, mentioning the fact that the experiments of C. P. Rumpf, M. Am. Soc. C. E., and of Mr. Forrest Nagler, as well as those of the writers, establish the reliability of these phenomena. The curves for the Pegram screw type of meter described by Major Somervell also showed this same characteristic, which was likewise observed by the writers on other meters with propellers of the screw type. The phenomenon seems to result from some unsymmetrical interference of the current leaving the rotating propeller with the frame of the meter. For some meters the magnitude of the deviation was different at different velocities. Local disturbance, as suggested by Mr. Leach, may possibly have been responsible for some of the deviation, although tests performed with current flowing in the main canal (which was 10 ft. wide) also showed similar differences between the registration of currents from the right and left direction. However, many tests in which screw meters have been towed in still water, have shown no great deviation.

TABLE 14.—PHYSICAL CHARACTERISTICS OF CURRENT METERS USED IN TESTS

Name of meter	Meter number	Fig.	TYPE OF PROPELLER			Number of revolutions per contact	Linear* distance for one revolution of propeller, in feet	Outside diameter of propeller, in inches	Distance, center of support to center of wheel, in inches	Direction of rotation of wheel‡
			Screw or cup	BLADES						
				Number	Kind					
Hoff	9	3(*a*)	Screw	3	Rubber	10	0.94	....	......	Clockwise
Hoff	9	3(*b*)	Screw	4	Rubber	10	0.94	4.00	......	Clockwise
Ott-Small..................	2914	4(*a*)	Screw	3	Fan	50	0.41	2.30	4.59	Counter-clockwise
Ott-Medium...............	2908	4(*b*)	Screw	2	Conical	25	0.85	4.79	7.43†	Counter-clockwise
Ott-Mensing...............	3246	5(*a*)	Screw	2	Conical	25	1.71	6.80	12.10†	Counter-clockwise
Ott-Mensing...............	3246	5(*b*)	Screw	3	Cylindrical	25	1.72	7.04	11.75	Counter-clockwise
Haskell-Type B............	154	6	Screw	4	High pitch	1	3.02	7.77	13.15†	Clockwise
Haskell-Type B............	154	7	Screw	4	Low pitch	1	1.23	7.67	12.98†	Clockwise
Price-Small................	211880	2	Cup	6	Cones	5	2.23	5.00	3.90	Counter-clockwise
Price-Acoustic	19	1	Cup	6	Cones	10	2.24	5.00	0	Counter-clockwise
Price-Small improved,....	26295	8	Cup	6	Cones	5	2.24	5.00	......	Counter-clockwise

* At stream velocities of 2 ft. per sec.

† Distance measured to up-stream end of wheel

‡ Looking downward or down stream at meter

In response to requests for additional information with regard to the meters used by the writers, Table 14 is presented giving physical characteristics of the various meters.

Numerous tests in which measurements by current meters have been checked by flow determinations by other methods, have been made during the ten years, 1920 to 1930, in the hydraulic canal at the Laboratory of the State University of Iowa. When the conditions have been favorable and care has been taken to traverse the measuring section adequately, the measurement of flow with current meters has always proved exceedingly reliable. In connection with experiments on the flow through box culverts in 1922 to 1924, one hundred and ten discharge measurements were made by Mr. Yarnell of flows between 5 and 165 cu. ft. per sec., by means of a Pitot tube, the small Ott current meter, and a sharp-crested rectangular weir.

TABLE 15.—POINTS FOR OBTAINING THE MEAN OF A VERTICAL VELOCITY CURVE

Number of points in vertical	LOCATION OF POINTS IN THE VERTICAL, IN FRACTION OF TOTAL DEPTH	
	P. Tchebycheff	B. F. Groat
2	0.211825	0.211825
	0.788675	0.788675
3	0.146446	0.146446
	0.500000	0.500000
	0.853554	0.853554
4	0.102673	0.1127017
	0.406204	0.3709006
	0.593796	0.6290994
	0.897327	0.8872983
5	0.083751	0.0837513
	0.312730	0.3127293
	0.500000	0.5000000
	0.687270	0.6872707
	0.916249	0.9162487
6	0.066876	0.07920706
	0.288740	0.2475242
	0.366682	0.4158414
	0.633318	0.5841586
	0.711260	0.7524758
	0.933124	0.9207929
7	0.058069	....
	0.235672	....
	0.338044	....
	0.500000	....
	0.661956	....
	0.764328	....
	0.941931	....
9	0.044206	....
	0.199490	....
	0.235619	....
	0.416047	....
	0.500000	....
	0.583953	....
	0.764381	....
	0.800510	....
	0.955794	....

More than one-half the comparisons showed less than 2% variation between the measurements by the weir and those by the current meter, and most of the measurements that showed a greater departure were performed under conditions not entirely ideal for the best current-meter measurement. However, in presenting the paper, the writers decided that it was advisable to compare the various meters purely on the basis of their registration of current velocity, rather than to compare the measured quantities of flow. In the latter case the location and number of metering points, the method of computing the mean velocity, and the measurement and method of computation of area of cross-section, often introduce other possible errors of no small magnitude.

The writers were greatly interested in Mr. Groat's discussion of rules for obtaining the average velocity in a vertical plane from velocity readings at definite points. The analysis and results are similar to those suggested by Patnutij Tchebycheff. The work of Tchebycheff was brought to the writers' attention in 1925 by A. Streiff, M. Am. Soc. C. E., in an unpublished paper entitled "The Use of Tchebycheff's Formula in Current Meter Measurements." The formula has been derived by the Naval Architect, J. H. DeHeer.[40] A comparison of the ordinates derived by Mr. Groat (see Table 9) with those obtained by Tchebycheff is given in Table 15. The results for two, three, and five ordinates are identical. The writers have tested the Tchebycheff rule on a number of mathematical curves and generally have found sufficiently close agreement to the true average for accurate practical work.

To average the measurements made by two different types of meters (the Ott and the Price, for example) appears now (1931) to be the "only rational method" of obtaining a reliable measurement of the discharge of turbulent water. This suggestion made by Mr. Leach follows the proposal[41] made by Mr. Groat, and has been specified in the A. S. M. E. Power Test Code, for measuring water by current meters in the testing of hydro-electric plants. The curves presented by the writers are useful in the determination of the comparative over- or under-registration of the meters being used. In extremely turbulent water, the use of a single meter does not appear feasible until some method is devised for determining an index of the relative degree of turbulence.

---

[40] *de Ingenieur,* November 14, 1908; see, also, *Hütte.*

[41] *Transactions,* Am. Soc. C. E., Vol. LXXX (1916), p. 1231.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1779

RELATION BETWEEN RAIL AND WATERWAY TRANSPORTATION: A SYMPOSIUM[1]

PAGE

Waterway Transportation from the Viewpoint of Operation.
By T. Q. Ashburn, Maj. Gen., U. S. A. 862

Railways Versus Waterways: An Economic Comparison.
By L. D. Cornish, M. Am. Soc. C. E. 877

Relation Between Rail and Waterway Transportation.
By E. A. Hadley, M. Am. Soc. C. E. 898

With Discussion by Messrs. Lewis C. Sorrell, R. B. Kittredge, Henry A. Palmer, J. Parker Snow, Baxter L. Brown, Theodore Brent, Samuel O. Dunn, Henry W. Hobbs, C. W. Kutz, T. Q. Ashburn, L. D. Cornish, and E. A. Hadley.

---

[1] Presented at the Technical Session, Annual Convention, Milwaukee, Wis., July 10, 1929.

WATERWAY TRANSPORTATION FROM THE VIEWPOINT OF OPERATION

By T. Q. Ashburn,[2] Maj. Gen., U. S. A.

Synopsis

This paper gives a discussion and comparison of private, contract, and common carriage, as well as common carriage under joint rail and water rates, on three types of inland waterways: Lake type (Great Lakes and connecting channels), per ton-mile cost, 0.64 mill; lock-canal type, closed by ice part of year (New York Barge Canal), per ton-mile cost, 3 mills; and river type, adequate channel open the year round (Mississippi River, St. Louis, Mo., and South), per ton-mile cost, 1.07 mills.

The expense and difficulty incident to the development of, and also the benefits to the general public from, these types of water carriers increase in the order in which they are mentioned.

The development of private and contract carriage on the waterways creates monopolies which destroy healthy competition. To prevent this, common carriage should be developed, and the benefits of this type should be extended to communities off the waterways through the establishment of joint routes and rates in which both water and land carriers participate.

The Inland Waterways Corporation is an agency set up by Congress to develop joint land and water common carriage. Analysis of the Barge Line dollar shows a deficit of 3 mills in 1927 and a net income of nearly 6 cents in the succeeding year. The Corporation has demonstrated that it can operate on freight rates which give a substantial saving under all rail rates and, at the same time, by fair divisions of the revenue afford each participating carrier a living, thus proving that the Nation's policy to render its streams navigable in order to reduce transportation costs is not a mistake.

From Noah's time to this—a lapse of thousands of years—every man who throws himself whole-heartedly into the struggle for inland and coastwise waterways transportation meets the same objections and criticisms that old Noah met; but they have continued determinedly, and time and experience are dumfounding their critics, even as Noah did when his ark, with its precious burden of freight, rose upon the crest of the waters and saved the faithful few of those days, who believed in water transportation.

Expansion of Civilization and Transportation

The expansion of civilization is coincident with the expansion of transportation. Wherever civilization may have first appeared, it spread solely through contact with the outside world; and only as fast as such contact could be made. First, such contact was made by means of the legs of the human being; then through the animals he trained to support him; then through the slow-running vehicle, developing into the railroad trains, the automobiles, and the airplanes of to-day. The log that drifted down stream, the sluggish flat-bottom raft propelled by poles, developed into the ocean racers and the marvelous towboats of to-day.

---

[2] Chairman and Executive, Inland Waterways Corporation, Washington, D. C.

In its ultimate analysis, all communication between individuals, between communities, between States and nations, is some form of transportation. Speech is the transportation of sound through the air; the telephone, the telegraph, the wireless, the heliograph, all depend upon the transportation of sound or signals; and as civilization expanded, its onward course was marked and limited by the development of transportation.

With the growth of civilization began the growth of trade and commerce. As these took the form of water carriage and venturesome voyages, the necessity for agreements between neighboring sea cities became apparent—harbors and lighthouses had to be constructed for the safety of trade, and thus began that slow growth of mutual agreement or mutual accommodation, which is the basis of whatever international law exists.

Necessity Develops Transportation

Transportation brings from one end of the world to the other the products of Nature and of Man, and as the necessity of these products grows, be they raw or manufactured, so must transportation facilities advance.

Civilization brings in its train the demand for necessities, comforts, and luxuries, in the order named. Nature has ordained that the raw material needed in the manufacture and production of necessities, comforts, and luxuries should grow in widely separated portions of the globe. Certain kinds of grain, rice, and sugar cane, grow only in certain localities; various kinds of ores are produced at one place and turned into steel and iron at another place. The silks of China and Japan are as actual necessities to American women as are the farming implements of America, the products of great steel industries, the locomotives, in fact, all American exports, to South America, Europe, Asia, Africa, and Australia.

The laws of economical production demand that great manufacturing centers shall be located where cheap raw material can be obtained, and where the cost of distribution of the manufactured article is cheapest, provided there is no great difference in labor costs. A combination of these two elements, where the raw material is an import, naturally suggests a seaport. If the distribution can also be made from that seaport, so much the better. The sea is a universal highway—the cheapest means of transportation known. The reason for this is self-evident.

Consequently, every inland city, located upon a navigable waterway, dreams of the time when it shall be an inland seaport. If navigable highways are developed as they should be, it will only be a matter of a few years until such cities as the "Twin Cities" of St. Paul and Minneapolis, Minn., Cincinnati, Ohio, Pittsburgh, Pa., St. Louis and Kansas City, Mo., Omaha, Nebr., Montgomery, Ala., Albany, N. Y., Cairo, Ill., Memphis, Tenn., Birmingham, Ala., Vicksburg, Miss., and many others will be actually, if not geographically, seaports. In a somewhat limited sense the Great Lakes cities are already seaports.

THREE DISTINCT PROBLEMS

The navigable inland waterways of the United States, from an operating standpoint, naturally fall into three distinct classes, the Great Lakes, improved rivers, and canals. An examination of the required investment in floating equipment, physical conditions of operating channel, terminals, and overhead, necessary for successful operation upon these various water highways, is profitable regardless of the cost of creating these channels.

Since no charges are made for the utilization of any of these waterway avenues, the practical operator is not concerned with their cost of construction, nor is he inclined to view with alarm any enormous expenditures made, so long as they have been made, and so long as all the avenues are open to him without cost.

He accepts the situation as it is, and leaves to economists, statisticians, and college professors the academic question of whether Congress was right or wrong in spending money for such purposes. He says, in effect,

> "The Government may have been right or wrong in spending money to create these channels, but so long as the money has been spent, it has been spent, and academic discussion does not concern me; what does concern me is whether I can profitably operate upon the channels which have been created by the Government by the expenditure of such funds."

PHYSICAL CONDITIONS

The amount of money to be expended for the creation of a successful single unit for operation depends largely on physical conditions of operations. The Great Lakes are open for navigation approximately eight months of every year. The New York Barge Canal rarely has more than seven full months of navigation, but for comparative purposes it will be credited with eight. The Upper Mississippi, extending from the "Twin Cities" to St. Louis; the Missouri, from Kansas City to St. Louis; and the Chicago-Illinois River route are ice-bound approximately 3½ months per year. On the other hand, the Lower Mississippi is open the year round for navigation, as is the Warrior River. Ice obstructions on the Ohio are negligible during the entire year.

It is evident, therefore, that successful operation upon the Great Lakes, the New York Barge Canal, the Upper Mississippi, the Illinois-Chicago River route, and the Missouri, requires the accumulation of enough profits in eight months to make a reasonable return upon the investment for a whole year, or that some other use of the equipment must be found during the closed season.

The closing of the Port of St. Louis has no appreciable effect on the revenues of the Federal Barge Line, as joint rates remain effective from St. Louis *via* Cairo (the northern port when St. Louis is closed); and the shipper is not even cognizant that the river may be ice-bound from St. Louis to Cairo.

LIMITS OF DISCUSSION

In discussing the Great Lakes it is necessary to consider the canals and rivers connecting them as an integral part of one system. These rivers and canals being necessary for inter-lake communications, and connecting a series of navigable lakes extending approximately 1 000 miles east and west, can not be considered therefore in the same category as the New York Barge Canal, which, in turn, is different from the Dismal Swamp Canal.

Exception might be taken to thus including the Welland Canal in treating connecting lake canals as integral parts of the Great Lakes System, because it is Canadian owned; but that does not affect operating conditions, except for tolls, and the United States could easily connect Lakes Erie and Ontario as cheaply as Canada did. Without entering into any argument relative to the merits of the all-American route *via* the New York Barge Canal to the sea *versus* the Canadian route, it is feasible to discuss actual existing physical conditions, and demonstrate therefrom how utterly different are the various costs of lake, river, and canal transportation.

In discussing canals, the New York Barge Canal will be taken as typical, because it is the longest and best equipped canal in the United States, and of the greatest potential National importance. The operating costs there will be higher, of course, than on a canal free from locks.

Discussion of rivers will be confined to the Lower Mississippi, which is open for twelve months each year, and is, so far as operations are concerned, the most available in the United States.

One point needs emphasis—this discussion being solely to determine comparative costs deals only with the best existent type of channel in each case. Further, it is assumed that certain conditions, which do not now exist, will eventually exist, in order to arrive at what will be the eventual cost per ton-mile when the situation is practically perfected.

TYPE OF EQUIPMENT DISCUSSED

The many lake lines have various equipment, depending upon the service in which they are engaged. At least one line has self-propelled barges capable of navigating both the Great Lakes and the New York Barge Canal. Generally speaking, self-propelled barges for bulk cargoes on rivers and canals are uneconomical units. The New York Barge Canal and the Mississippi-Warrior Service also have different types of equipment, hence, to obtain comparative costs, an operating season for the most practicable single unit for each of these transportation lanes, will be studied. Such a comparison will assume the most favorable conditions possible for the three separate water avenues.

On the Great Lakes, take a 16 000-ton steamer having an average speed of 10 miles per hour, plying between Cleveland, Ohio, and Duluth, Minn., and capable of carrying coal from Cleveland to Duluth and iron ore in the reverse direction. Its cost would be approximately $1 500 000. Return on this cost at 6% plus 3% depreciation will be $135 000 per annum; this is the amount that must be earned in 8 months to yield a reasonable return.

On the New York Barge Canal, assume a unit consisting of a self-propelled barge, capable of carrying 300 tons of cargo, and pushing seven barges of

500 tons capacity each at an average of 5 miles per hour. This rate is used because a greater speed causes a very expensive wave wash, entering into cost of canal maintenance. Such a unit as used by the most efficient organization now operating on the canal, could be built for about $400 000. Return and depreciation, as before, would here equal $36 000 per annum to be earned in 8 months to offer a reasonable return.

On the Lower Mississippi, assume a towboat and 8 barges capable of carrying 16 000 tons down stream, costing $650 000. The average speeds will be down stream 6 miles per hour and up stream 3 miles. The return of $58 500 per annum (9%) may require 12 months and still offer reasonable earnings.

Types of equipment actually used on the Mississippi River are shown in Figs. 1 and 2, including the method of towboat and barges taken as a basis for comparison.

Effect of Seasonal and Channel Differences

A lake steamer can carry the same tonnage in either direction, at about the same speed, without difficulty. The Barge Canal unit can never carry up stream the same tonnage as down stream, because there is always some current in the Mohawk, and sometimes it appreciably affects the carrying capacity. Normally, it should carry three-quarters as much.

The Lower Mississippi always has some current. A towboat which will successfully push 16 000 tons down stream at a certain speed can rarely make the same speed up stream with only one-half that total. Even then the cost of fuel up stream for the half cargo is approximately double that for the maximum, down stream.

Neither the Great Lakes System nor the Barge Canal will be subject to low water, but, at times, the canal will have to contend with very high water and strong current, while the Mississippi will be subject to such changes of high and low water that the carrying capacity of the fleet will be a variable. All these factors, it will be observed, will have an appreciable effect on the carrying cost per ton-mile of each unit.

The direct route varies in each instance. From Cleveland to Duluth by lake, it is 810 miles; from Buffalo to Troy, N. Y. by canal, it is 341 miles and has 36 locks; from St. Louis to New Orleans by river it is 1 154 miles.

In order to strike a fair comparison the further assumption is made that the lake, canal, and river cargoes are delivered to the various units at one port of call. By thus reducing these operations to the simplest possible terms a definite cost basis results, which is fair to all concerned; but even with this condition, due to the necessity of shifting barges for loading and unloading, the lake ship will save at least a day loading and a day delivering the same cargo.

Comparative Round Trip Costs

It will be instructive to follow the lake steamer. Exclusive of interest and depreciation, it costs about $325 per day to operate. Allow it 2 days at Cleveland for loading, 2 days at Duluth to unload, 2 to reload, and 7 days en route, or 13 days for the round trip. That would permit 19 round trips during the season of 8 months.

The actual running expenses for this would be 245 × $325 = $79 625. With the necessary interest and depreciation, this would give a total of $214 625

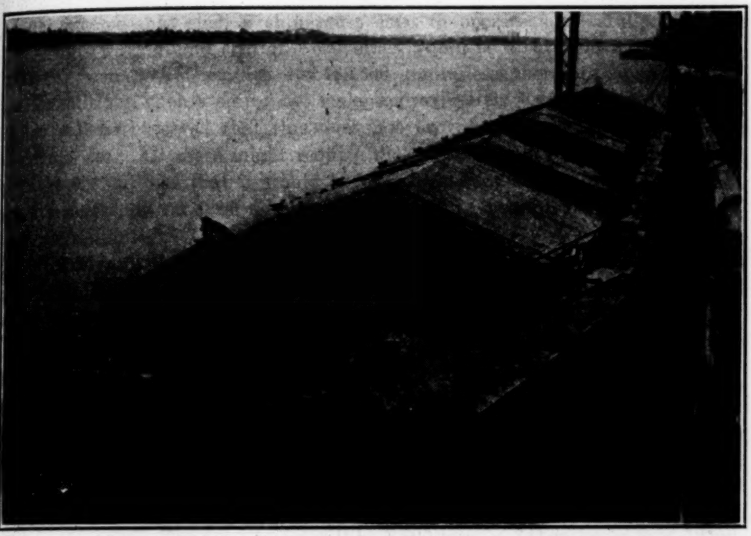


FIG. 1.—STEEL BARGE WITH 70 000 BUSHELS OF GRAIN, INLAND WATERWAYS CORPORATION.

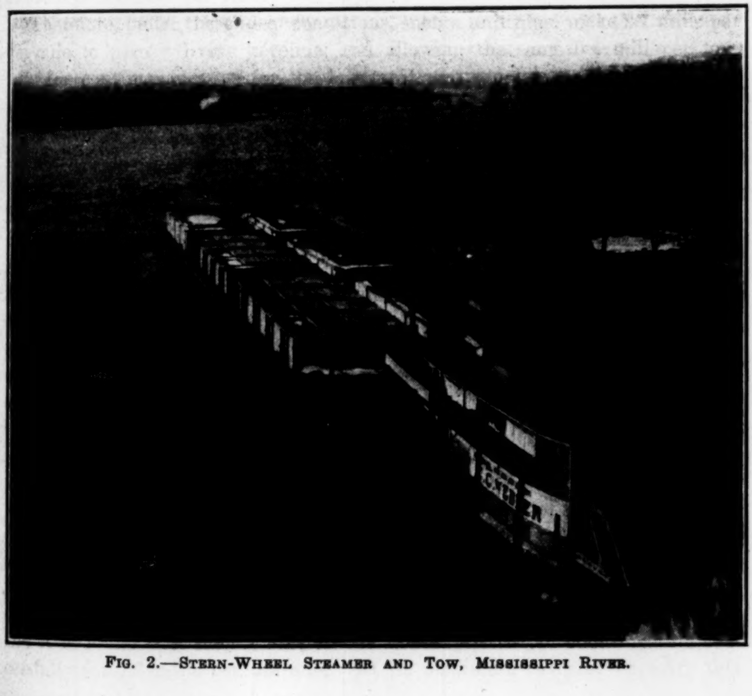


FIG. 2.—STERN-WHEEL STEAMER AND TOW, MISSISSIPPI RIVER.

annual income that such a ship must earn to operate successfully. Since 16 000 tons for a round trip of 1 620 miles would be 25 920 000 ton-miles, 19 trips per year would require 492 480 000 ton-miles; therefore, under these ideal conditions, such a ship must make approximately 0.44 mill per ton-mile, but it will be observed that this does not provide for overhead, insurance, accidents, etc. All these might require the addition of 0.4 mill per ton-mile, possibly 0.2 mill; so that it would be practicable for such a unit to operate quite successfully on 0.84 mill per ton-mile and, theoretically, even at 0.64 mill per ton-mile.

Similarly, on the New York Barge Canal, a unit would need 3 to 4 days at Buffalo to load, 6 to 7 days to unload and load at New York, 4 to 5 days down stream, and 5 to 6 days up stream, or an average round trip of 20 days. Lockage difficulties and delays account for these variables, allowing a total of 12 round trips per unit per season. Operating expenses of such a fleet aggregate roundly \$192 per day. The actual expenses for this time would be (245 × \$192) + \$36 000 = \$83 040, which is the actual return necessary to operate successfully.

Assume the load to be the same each way, although that up stream will normally average only three-quarters of that down stream, and the speed up stream will vary. The total traffic will be 3 800 tons × 341 miles × 2 × 12 = 31 099 200 ton-miles per year.

Therefore, under these ideal conditions, such a unit must make 2.7 mills per ton-mile to earn a living revenue, and allowing the same 0.4 mill per ton-mile for contingencies, as for the lake unit, the canal unit can operate successfully on 3.1 mills per ton-mile. Actually, its contingent expenses ought to be covered by 0.3 mill per ton-mile, so, theoretically, it ought to operate successfully for 3 mills per ton-mile.

On the Lower Mississippi, with the same time allowance as for the lake unit, namely, 6 days for loading, unloading, and reloading, with a round trip, St. Louis to New Orleans, of 2 308 miles, and with an average speed of 4 miles per hour, the actual running time would be 24 days. This would give 30 days per trip and 12 trips per annum. It must be assumed under general conditions, that the maximum down-stream tonnage will be 16 000, while the up-stream maximum will be 8 000, giving an average of 12 000 tons. River conditions, however, as shown by Government records, would reduce the yearly average to 6 000 tons per trip. Both high and low water appreciably affect the tonnage and speed.

The actual expenses of this operating fleet would be \$430 per day, and the total for the year would be \$430 × 365 = \$156 950. To this must be added the carrying cost of \$58 500 per annum, making a total of \$215 450 necessary to yield a reasonable return. Then, 6 000 tons × 1 154 miles × 2 trips each way × 12 round trips = 166 176 000 ton-miles; and \$215 450 ÷ 166 176 000 = 1.3 mills per ton-mile. Add to this the same 0.4 mill per ton-mile to cover expenses not otherwise provided for (this is the actual cost on the Mississippi), and it is found that such a unit on the Lower Mississippi could operate successfully for 1.7 mills per ton-mile. At its maximum carrying capacity this

0.4 mill per ton-mile could easily be reduced to 0.2 mill, and successful operation be carried on at 1.5 mills per ton-mile. The direct operating cost would also be reduced.

Summing up, a theoretically successful operation by lake could be carried on with an operating income of 0.64 mill per ton-mile; on the Lower Mississippi, with one of 1.7 mills per ton-mile; and on the New York Barge Canal, with one of 3 mills per ton-mile.

While the difference in cost under these assumed ideal conditions on the Great Lakes and the Lower Mississippi is 1.06 mills per ton-mile, yet when this is applied to the 166 176 000 ton-miles on the Lower Mississippi, it adds a total of $176 146.56 per annum to the gross income necessary to operate at a fair profit.

Such an itemized estimate of the theoretical incomes necessary to operate successfully upon the three types of lake, canal, and river channels under ideal conditions, is necessary because in most public discussions ardent waterway advocates point to lake costs as the ultimate cost by river, and the opponents of waterways point to the Erie Canal as the great failure whereby river transportation should be judged. As has been shown, neither of these is a correct assumption; nor is it correct to group the three together as a whole. So much for the ideal situation. The next question concerns the actual conditions under which rivers and canals may be utilized.

PRIVATE, CONTRACT, AND COMMON CARRIERS

Lakes, canals, and navigable rivers can be utilized by private, contract, or common carriers. The cost of operation of these three carriers, and their value to the public, increase in the order named.

The utilization of navigable streams by private and contract carriers alone does not justify the expenditures of the money of all the people for their creation. Indeed, if such utilization were the sole purpose of public construction, a large part of the people would be unjustly taxed for the benefit of a few who would reap the advantages.

Any thinking man must realize that the utilization of streams by private and contract carriers alone may lead to serious consequences, so that to extend fully the benefits to all the people who have been taxed to create them, to offset the serious menace to National prosperity that such utilization alone may lead, these private and contract carriers must be supplemented by powerful and efficient common carriers. This is not to be assumed as decrying the fullest possible utilization of streams by private and common carriers; rather it will be pointed out how the menace of such utilization can be absolutely neutralized by powerful and efficient common carriers, and how such joint utilization will result in the greatest good for the greatest number.

The private carrier requires the least organization and operates purely by water. It can balance its freight savings against the cost of operation and if the freight savings are the larger of the two, it can be said to be a successful carrier.

The contract carrier may operate purely by water or by water and rail. It has comparatively small overhead. As a water operator, pure and simple, it has no offset against its cost of operation, because the freight saving does not accrue to it, and it has to have a gross income larger than its gross expenses to operate at all. If it has any relations with railroads, they are limited and simple, and it avoids the tremendous overhead of the common carrier.

If the common carrier is a carrier by water only, it still has expenses which a private or contract carrier does not have, because it needs organizations at each port touched by it. The private or contract carrier may only have one port of origin and one of destination.

If the common carrier by water fulfills its highest function of serving all the people, it must have a large overhead, and this overhead is fixed by its activities, not by the size of its fleet. In other words, the overhead of such a common carrier is scarcely any greater if the fleet be doubled.

VALUE OF THREE TYPES

One of the dangers of the utilization of navigable streams and canals only by private carriers which are big corporations, is that the construction of the fleets necessary to carry the tremendous bulk essential to economical operation involves such a large original investment of capital as few corporations can afford. Hence, those fortunate enough to be able to create and operate their own fleets, can transport their products cheaper than their competitors and undersell them, thus destroying healthy competition.

A contract carrier may be of great advantage, or a positive detriment, to a large portion of the country. To illustrate, if coal operators in Southern Illinois owned their own facilities for transporting coal cheaply to Minnesota, and did so, and sold it cheaper, every consumer of heat, gas, light, and other power served by the "Twin Cities" would be benefited. If such coal operators, in addition, contracted to carry back ore from the Mesabe Ranges in Minnesota, their cost of transporting their own coal would be cheapened, the user of ore for manufacturing purposes would be benefited, and if the manufactured article bore a proportionate decrease in price to the decreased transportation costs, vastly more people would be benefited.

On the other hand, suppose a contract exists between certain grain dealers in, say, Kansas City, Mo., and Omaha, Nebr., a railroad connecting with St. Louis, which owns its own elevator, and a contract river carrier. If this railroad guarantees to maintain its elevator full enough of these particular operators' grain to keep the water carrier busy, and if the water carrier transports only for these particular operators, the combination can practically set the purchase price of grain in Kansas City and Omaha. Not only can it cripple all other grain operators who have to depend upon all-rail rates, but it can compel the farmer who sells to their competitor to accept a lower price for his grain, because he has to deduct from his purchase price the higher cost of transportation. However, if there exists in competition with such private and contract carriers, a great common carrier, offering to all the people the same facilities that the others offer to a few, and at the same price, there is no chance of the private and contract carriers creating a monopoly that will throttle competition.

Broadly speaking, the conditions precedent to the success of a great common carrier are not necessarily present for the success of a private or contract carrier. At the same time, to extend the benefits of cheaper water transportation on inland rivers and canals to all requires the utilization of great common carriers. To create these conditions precedent to success it has been found necessary to have a governmentally owned corporation, functioning as a private corporation, but backed by the great powers of the Government itself. Railroad opposition had been successful in practically annihilating common carriers on navigable streams and lakes.

A private carrier requires:

(*a*) A suitable navigable river.
(*b*) Suitable boats.
(*c*) Terminal facilities, which may be very simple, suitable for handling only one commodity.
(*d*) A small operating force.

A contract carrier requires, in addition:

(*a*) Terminal facilities, which are more complicated than those necessary for handling one commodity only.
(*b*) An operating force probably one-half greater than a private carrier.

A common carrier, to be of nation-wide benefit, must have in addition to the requirements of both the private and contract carrier:

(*a*) Balanced freight.
(*b*) Terminal facilities, dependent upon conditions in each locality both ashore and afloat; and more of them.
(*c*) Interchange with other carriers.
(*d*) Equitable division of revenue accruing from joint rail-water haul.
(*e*) A very large operating, traffic, and auditing overhead.

What follows refers to a great common carrier of nation-wide benefit, and not to a common carrier by water alone. These latter are highly desirable, but of no great National benefit.

A great common carrier must be organized along the lines of good railroad practice, must have as keen a traffic and operating manager, and other officials, as any railroad organization, and must be as closely co-ordinated through an executive as are the various divisions of a railroad. In order to be successful, it must also have much and costly equipment and powerful financial backing.

A demonstration by the present fully empowered governmental body (Inland Waterways Corporation) has been considered as necessary by Congress until such time as conditions precedent to success are established and private capital is willing to invest in an operation no longer hazardous. From this standpoint the experience of the Corporation with its Barge Line is instructive.

INLAND WATERWAYS CORPORATION OPERATIONS

Freight transported by common carriers is carried under what are known as class and commodity rates. These class rates vary considerably, for example, the first, second, third, fourth, and fifth class rates *via* the Barge Line from Minneapolis to East St. Louis, Ill., are, in cents per hundred pounds, 77, 63½, 51, 36½, and 26; whereas the similar rate on export grain from St. Louis to New Orleans is only 11½ cents.

Relative to the actual operations of the Inland Waterways Corporation, Table 1 provides a basis upon which calculations regarding the economics of water transportation may be made. Its practical value is enormous, and

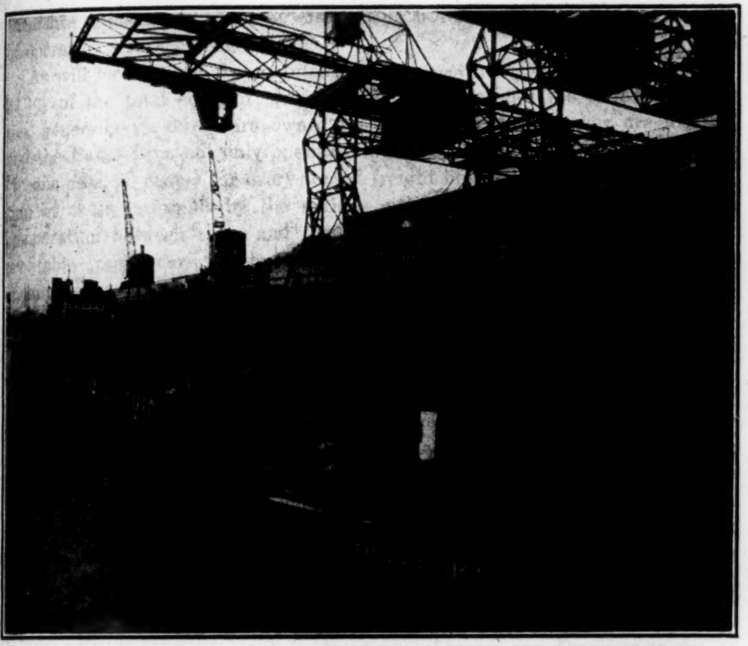


FIG. 3.—SELF-PROPELLED BARGE, MUNICIPAL WHARF, ST. LOUIS, MO., SHOWING METHOD OF LOADING AND STOWING FREIGHT.

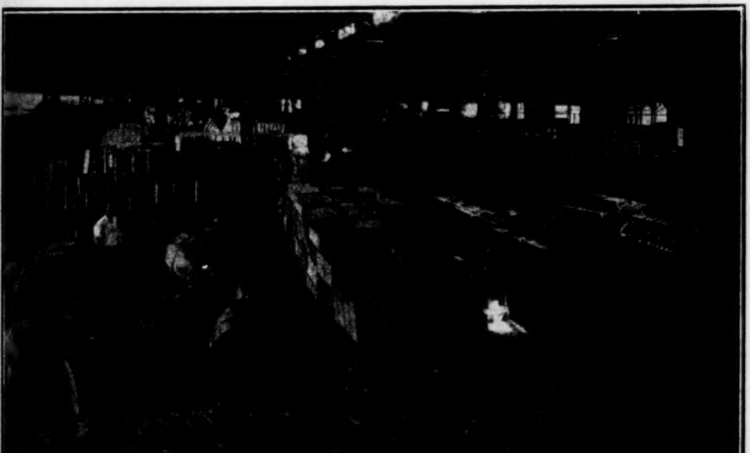


FIG. 4.—INTERIOR OF BIENVILLE STREET WHARF, NEW ORLEANS, LA., SHOWING CLASSIFICATION OF FREIGHT.

furnishes a mark "to shoot at" in getting a reasonable ratio between costs of operation and accruing revenue.

As will be observed the maintenance and operation of terminals comprise 39.3% of the total costs of operation. That is because the terminal facilities, generally speaking, are owned by the cities in which they are located, and the Barge Line not only operates and maintains them, but pays 15 cents (in one case, 20 cents) for every ton of freight handled over them. The erection of these terminals by the various cities was essential to promote that co-operation between them and the Barge Line so necessary to build up a profitable freight-carrying business. Of course, the city is stimulated and encouraged to utilize the terminals as much as possible to get a greater revenue, and this 15 cents per ton is supposed to be sufficient to take care of the interest on the bonds, pay taxes, and amortize the investment. For example, in 1928, the City of Memphis received an approximate total of $50 000 for the use of its terminal, and the Barge Line kept it in repair. As traffic increases, this annual payment increases. This is the answer to the public statement so frequently made that the tax-payer is being mulcted by the Barge Line to pay for terminals. The amount of local freight that passes over these terminals is small compared to the interchange freight, and while the city is being reimbursed for its original expenditures, it has been helping those in the hinterland to save money on freight bills, and thus increase their purchasing power, which purchasing power is directly reflected in the city's prosperity.

TABLE 1.—THE BARGE LINE DOLLAR AS INCOME AND DISBURSEMENT.

(Calculated on the total revenues and total expenses of the Operating Divisions of the Inland Waterways Corporation.)

	1927.	1928.
Where the Dollar Came From:		
Total revenues	$0.997	$1.00
Operating deficit covered by Corporation	0.003	......
Total	$1.00	$1.00
Where the Dollar Went:		
Maintenance and operation, Line vessels	$0.439	$0.408
Maintenance and operation, terminals	0.407	0.393
Traffic expenses	0.045	0.041
General expenses	0.045	0.049*
Loss and damage	0.032	0.029
Maintenance and operation, railroad	0.021	0.021
Charter expenses	0.011	0.005
Net income	......	0.059
Total	$1.00	$1.00

* The increase in general expenses of 1928 over 1927 of 4 mills in the dollar was due to: (*a*) Higher cost of insurance due to tonnage increase and greater protection; (*b*) salaries and expenses of upper river officials; and (*c*) increased office rent and expenses at New Orleans.

Originally, no attempt at standardization of the type of terminals was made, nor could it be made. There were no standard barges, and no data available as to the expense of operating, so that in the existing system may be found the direct-lift, escalator type, and the inclined railroad with floating terminal. In the operation of terminals there is thus an essential difference between rail and water transportation. Figs. 3 and 4 show the appearance and the operation of two of the terminals in use on the Mississippi River.

Interior water transportation has inherent advantages and equally inherent disadvantages when it comes to the question of reflecting savings to those not fortunately located on navigable streams. The physical nature of waterways limits the ports at which tows may touch or cargo be received, but rail or motor transportation has no such limitation. There thus results in Barge Line operations a costly transfer from car or truck to barge, or *vice versa*; and sometimes a second costly transfer, where the freight originates on a rail line, is transferred to barge, and again to rail as, for example, on a shipment from Chicago to Houston, Tex. Here, the freight moves by rail to St. Louis, is transferred to barge, and re-transferred from barge to car at Baton Rouge, La. These costs of transfer must be absorbed in rail-water-rail transportation; and still the resultant cost of the movement must be less than the all-rail movement. Otherwise, there is no virtue in such a form of transportation.

A shipper by rail can put his freight in a car at Chicago, and it will move through to destination without change of container, so that by a joint rail-water-rail route two transfers are introduced which are unnecessary in all-rail movements, but which cannot be obviated in joint movements. When the point of origin and the destination are river ports, and the same barge can carry its cargo from point of origin to point of destination then, and then only, can water transportation offer a comparable service to an all-rail service.

If it be necessary, as it now is, to transfer cargo moving between upper and lower river points, there is introduced a burdensome cost. Good business practice demands that this should be eliminated, but it could not be done as long as one fleet was designed to operate on a 4-ft. channel and the other on a 9-ft. channel.

It is to be noted that the costs of these transfers must be absorbed in the joint rail-water-rail rates, lower than all-rail rates, or the public will receive no advantage from such joint rates.

A Successful Policy

The whole National policy of rendering streams navigable, upon which so much money has been spent and will be spent, was based on the hope that by their utilization cheaper transportation would result. The Inland Waterways Corporation, a Congressional agent, was organized for the purpose of demonstrating that this policy is not a colossal failure.

It safely may be said that the demonstration has convinced the people of the United States that the policy of Congress, far from being a great mistake, has been the result of far-sighted vision; and the people themselves have demanded further and further expansion of the operations of this Corporation. They have seen its practical effects—cheaper transportation for all through co-ordination of rail, water, and motor transportation, with an accruing revenue to the joint carriers which, when fairly divided, will afford each participating carrier a reasonable living revenue.

RAILWAYS VERSUS WATERWAYS: AN ECONOMIC COMPARISON

BY L. D. CORNISH,[3] M. AM. SOC. C. E.

SYNOPSIS

The purpose in preparing this paper has been to assemble authoritative facts relative to past rail and waterway traffic in a form convenient for study and analysis so that conclusions or opinions relative to the economics of present and future railway and inland waterway transportation could be subjected to the acid test of a fair comparison with past records of freight transportation upon the railroads and waterways of the United States.

The essential railway statistics from 1890 to 1924, inclusive, were assembled in graphic form in 1926, and on the theory that the past trend is the most probable indication of the trend in the near future, the demands of transportation and railway requirements to meet them are forecasted to the year 1950. It is shown that the railways during the 25-year period must expend approximately $11 000 000 000 for freight capital cost of additional transportation facilities.

The cost of completing the Mississippi Valley Inland Waterways System and providing thereon floating equipment and terminals for 60 000 000 000 ton-miles of traffic is developed from known costs and shown to be $600 000 000. It is shown that equivalent railway facilities will cost more than three times as much.

The past growth of waterway transportation is developed and forecast to 1950, and it is shown that the completion and utilization of the Mississippi Valley System by that time will be of economic benefit to the amount of more than $90 000 000 per annum.

Forecasts and future requirements are based on statistics to 1925, but the graphs and tables are brought as nearly up to date as published data would permit and serve as a check and confirmation of the original predictions.

INTRODUCTION

The history of mankind shows conclusively that the predominating influence in the rise and fall of nations and cities has been the relative cost of transportation. The life of nations has been, and still continues to be, a life of warfare—if not the warfare of bloody battlefields, it is the bloodless warfare of trade and commerce, which begets real conflict.

---

[3] Chf. Engr., Div. of Waterways, State of Illinois, Chicago, Ill.

Bacon said: "There be three things which make a nation great and prosperous: A fertile soil, busy workshops, and easy conveyance for men and commodities from one place to another." The easy conveyance of commodities in these and future days means cheap transportation facilities. In the United States the ease of transportation is now measured by freight rates in mills per ton-mile.

In 1924 the railroads of the United States transported 392 000 000 000 ton-miles of freight at an average rate of 11.32 mills per ton-mile. In the same year the report of the St. Marys Falls Canal shows lake shipments of 72 000 000 000 ton-miles at 1.2 mills per ton-mile. In six years, 1923 to 1928, the Government Barge Line on the Mississippi River between New Orleans, La., and St. Louis, Mo., transported more than 6 000 000 000 ton-miles of freight for an average revenue of 3.90 mills per ton-mile. This economy of water transportation is impressive. Coupled with the appeal of the agricultural and manufacturing interests of the Middle West for lower freight rates to seaboard ports, it suggests the economic necessity of completing the improvement of the Mississippi River and its main tributaries for navigation purposes as soon as possible and of developing thereon a water-borne commerce at lower freight rates. Then, the products of the fertile soil of this region may compete in the markets of the world on a more profitable basis with similar products from Canada, the Argentine, and Russia. Then, the workshops of the region may again become busy manufacturing goods for the Pacific Coast and the Orient. Thus, they may regain customers lost to them for years past on account of the advantage of the low freight rates by water *via* the Panama Canal possessed by their Atlantic Coast competitors.

MODERN WATERWAY HISTORY

The Mississippi River, with its tributaries, comprises a natural waterway system more than 15 000 miles in length. About 5 000 miles, when the authorized improvements are completed, will constitute the trunk lines, reaching from Pittsburgh, Pa., to Kansas City, Mo., and from New Orleans to Chicago, Ill., and St. Paul, Minn. The area reached by these trunk waterways alone is so vast in extent, so rich in its products of agriculture, mining, and manufacturing, that the controversy over the relative economy of transportation by railroads *versus* inland waterways must inevitably be settled by actual operation with modern equipment upon modern waterways.

The movement for the development of the inland waterways of the United States appears to have run in cycles of about 20 years' duration, which have culminated in legislative authorization for the construction of one or more important waterways. During these cycles the persistent efforts of waterway advocates to educate the public and to secure legislative authorization for their projects have been as persistently opposed by interests which fear the competition of waterway transportation, or by persons who cannot, or will not, visualize the transportation needs of coming generations.

The waterway controversy which culminated about 1904 resulted in the construction of the New York Barge Canal and the Panama Canal. A barge, instead of a ship, canal across the State of New York was an economic mistake for which the selfish or misguided interests of the Cities of New York and Buffalo are solely responsible. At the time, waterway advocates and engineers were nearly unanimous in recommending a ship canal. This mistake has since been acknowledged by the effort of the State to have the Federal Government reconstruct part of the Barge Canal into a ship canal, instead of participating in the proposed St. Lawrence River improvement.

The Panama Canal has proved to be the wonderful military and commercial success which was anticipated by its advocates. Together, the high rail rates and the economic advantage which the Panama Canal gives to the Atlantic and Pacific Coasts are responsible for the unanimity of opinion of agricultural and manufacturing interests in the Middle West in favor of promptly completing the improvements of the Mississippi River and its tributaries for navigation purposes and of connecting them with the Great Lakes at Chicago. The cost of construction and maintenance and also the operating efficiency of the Lakes to the Gulf Waterway are dependent on a diversion of water from Lake Michigan at Chicago through the completed Chicago Sanitary Canal, thence through the Illinois Waterway, the Lower Illinois River, and the Mississippi River.

OPPOSITION TO WATERWAYS

A subtle, persistent, and effective opposition to the Illinois Waterway has existed for forty years among the interests naturally opposed to waterways in general and to this waterway in particular. The diversion question is limited to this particular part of the inland waterway system of the Mississippi Valley and may be dismissed as having only a minor influence on the relative economic importance of railroads and inland waterways in solving the future transportation problem of the United States.

The opposition to the development of waterways for transportation purposes is in the main from two classes, namely, railroad officials, or interests, and professional economists. The arguments of the railroad class are naturally biased by its concern as to the possible effect water transportation might have on its income and net earnings. Both classes are deeply perturbed at what they call the economically unsound and commercially unfair policy of developing waterways at public expense, and usually point out some particular waterway as a horrible example because of its small tonnage as compared with the large annual expenditures charged against it.

Conclusions relative to waterways in general are worthless if based entirely upon the results obtained from a study of isolated examples. Opponents of the improvement of waterways seldom, if ever, give any consideration to the total future transportation needs in such a rapidly growing country; nor will they admit the economic necessity of any section of the country being given lower freight rates than other sections in order to have a fair chance in competition for a profitable existence.

THE TRANSPORTATION PROBLEM

Railroads are and probably always will be the principal National agencies of transportation, but the rapidly increasing demand for transportation coupled with the periods of rail-traffic congestion and demoralization during past years, gives rise to several questions relative to the expected status of the National transportation problem twenty years hence, or, say, by 1950.

The principal questions are as follows:

(*a*) What will be the probable transportation demand upon the railroads of the United States in 1950?

(*b*) What will be the cost of additional railroad trackage and equipment required to supply the probable increase in transportation demand?

(*c*) How much of the transportation demand can be supplied by inland waterways now authorized, but uncompleted?

(*d*) What will be the probable cost of providing inland waterway transportation facilities, terminals, and operating equipment?

(*e*) What will be the relative benefit to the public for freight shipments by rail or inland waterways?

DEMAND AND COST OF INCREASED TRAFFIC

Railroad Traffic Data.—The questions as to the growth of transportation and its cost depend upon such a complexity of circumstances and conditions so difficult to forecast that the best solution lies in assuming that the trend of transportation progress in the United States during the past forty years will continue for the next twenty years.

TABLE 2.—UNITED STATES RAILROAD DATA PROJECTED TO 1950.

Curves.	Item.	Probable amount by 1950.	Reference, Interstate Commerce Commission reports.
A	United States population, in millions	150	
B	Traffic, all roads, in billions of ton-miles	870	1924, p. XCV
C	Freight density, in millions of ton-miles per track-mile	1.6	1924, p. XXXIX
D	Track length, total, in thousands of miles	555	1924, p. XCII
E	Length, main track, in thousands of miles	355	"
F	Length, yards and sidings, in thousands of miles	200	"
F'	Length, yards and sidings, in percentage of total	36	
G	Average capacity, freight cars, in tons	53	1924, p. XCII
H	Average load, freight cars, in tons	23	1924, p. XCVI
I	Travel of freight cars per annum, in thousand of miles	12·2	1924 pp. XCVI, XCII
J	Work of freight cars per annum, in thousands of ton-miles	281	
K	Total number of cars, in millions	3·1	1924, p. XCII
L	Aggregate car capacity, in millions of tons	164	
M	Average locomotive tractive capacity, in tons	31	1924, p. XCII
N	Total tractive capacity, in millions of tons	2·4	1924, p. XCII
P	Number of locomotives, in thousands	78	
S	Total capital cost, in billions of dollars	33·7	1924, p. XCIII

The Annual Reports of the Interstate Commerce Commission on the "Statistics of Railways in the United States" contain tabulated transportation statistics for all railroads and for each Class I railroad. In order to simplify the visualization of certain statistics the data have been plotted as shown in Figs. 5 to 8. In each case the solid lines show the data as given in the reports, and the broken-line continuations indicate the writer's opinion as to the probable future trend to 1950. The various graphs are lettered for convenience and the results indicating the expectancy in the several items by 1950 are given in Table 2, together with reference to the source of information.

In Curve *B* (Fig. 5) the prolongation of the mean curve answers Question (*a*) by indicating 870 000 000 000 ton-miles as the expected traffic by 1950. This is more than double the traffic of 1925, and it will undoubtedly require huge expenditures of capital to provide additional tracks, yards, and equipment. Curve *C* (Fig. 5) shows the density of traffic and indicates the average efficiency of the railroads with respect to their freight movement. A scrutiny of the records for individual roads develops an amazing variation in this measure of railroad performance and shows that an average of more than 1 000 000 ton-miles per track-mile is due to the coal-carrying roads of the Eastern and Southern Districts.

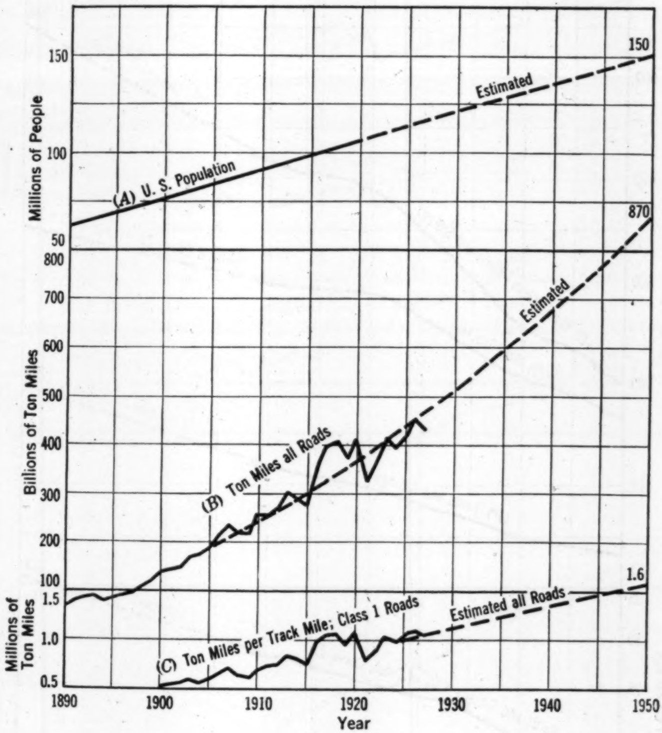


FIG. 5

Track Requirements.—Curves *D* and *E* (Fig. 6) were not directly extended because of the change of trend in recent years. New trackage requires heavy expenditures of new capital. During and immediately following the World War new capital was almost unobtainable, and the railroads were forced to provide for the unprecedented traffic demand by improving their operating efficiency. This resulted in a cessation of main-line track construction and disrupted the trend of Curves *D* and *E*. However, the construction of new track for yards and sidings had to be continued, as shown by Curve *F* (Fig. 6), and this provides a method for forecasting main-track requirements and for projecting Curves *D* and *E*.

Curve *F* shows a remarkably uniform trend of yard and siding construction which apparently is not much affected by periods of business depression or difficult financing. Therefore, this curve may be prolonged to 1950 (Fig. 6) with reasonable accuracy.

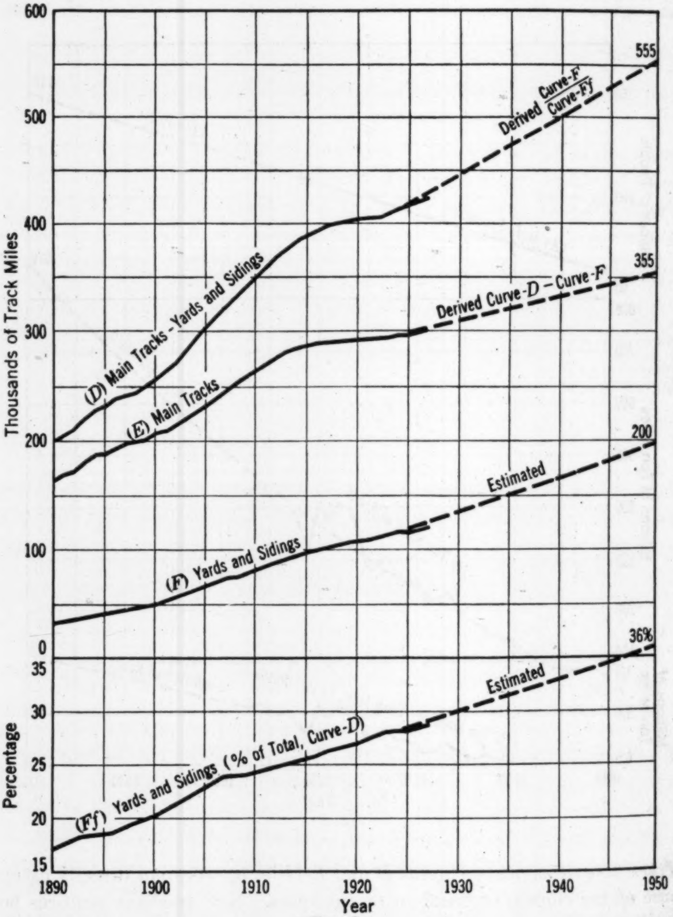


FIG. 6.

Curve *Ff* (Fig. 6), prepared to show what percentage of all tracks is in yards and sidings, also shows a steady increase. Prolonging its definite trend

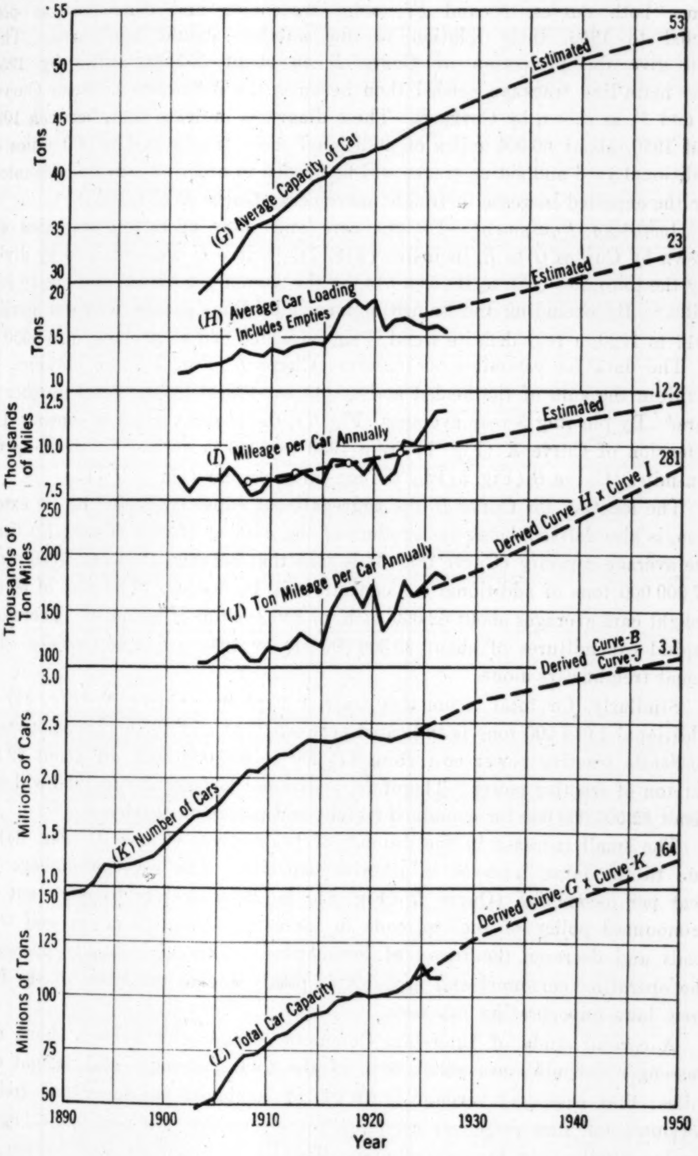

55
50
45
40
35
30
Tons
53
Estimated
(G) Average Capacity of Car
20
15
10
Tons
23
Estimated
(H) Average Car Loading
Includes Empties
12.5
10.0
7.5
Thousands of Miles
12.2
Estimated
(I) Mileage per Car Annually
250
200
150
100
Thousands of Ton Miles
281
Derived Curve H x Curve I
(J) Ton Mileage per Car Annually
3.0
2.5
2.0
1.5
1.0
Millions of Cars
3.1
Derived Curve-B / Curve-J
(K) Number of Cars
150
125
100
75
50
Millions of Tons
164
Derived Curve-G x Curve-K
(L) Total Car Capacity
1890
1900
1910
1920
1930
1940
1950
Year

FIG. 7.

indicates that by 1950 about 36% of all tracks will be in yards and sidings. Since both curves, *F* and *Ff*, seem reasonable and consistent as projected to 1950, their relation to one another should hold true. This will give the projection of Curve *D* to about 555 000 miles by 1950. The main-line trackage needed then becomes the difference between Curves *D* and *F*, as shown by Curve *E*. These diagrams indicate that, between 1925 and 1950, about 60 000 miles of additional main tracks and 80 000 miles of additional yard and siding tracks will be needed in order to provide adequately for the expected increase in freight movement (Curve *B* (Fig. 5)).

Operating Equipment.—Freight cars and their operation statistics are shown by Curves *G* to *L*, inclusive (Fig. 7). Curve *H* was obtained by dividing the ton-miles (Curve *B* (Fig. 5)), by the sum of the loaded and empty car-miles.[4] By excluding the known abnormal loadings of the post-war period, 1917 to 1920, a very definite trend is shown which can be prolonged to 1950.

The data for car-miles per annum (Curve *I* (Fig. 7)) are obtained by dividing the sum of the loaded and empty car-miles[4] by the total number of cars.[5] By plotting 5-year averages (Fig. 7), the general trend is shown. The extension of Curve *K* (Fig. 7) from 1930 to 1950 is derived by dividing the ton-miles (Curve *B* (Fig. 5)) by the ton-miles per car (Curve *J* (Fig. 7)).

The location for Curve *L*, the aggregate car capacity, including its extension, is also derived, being the product of the number of cars (Curve *K*) times the average capacity (Curve *G*). It is seen that between 1924 and 1950 about 57 000 000 tons of additional car capacity will be needed. The cost of 50-ton freight cars averages about $2 000 each, or $40 per ton of capacity. Therefore, capital expenditures of about $2 300 000 000 by 1950 are indicated for additional freight cars alone.

Similarly, for total locomotive tractive capacity (Curve *N* (Fig. 8)) an additional 1 014 000 tons is indicated between 1924 and 1950. Locomotives of 60 000-lb. tractive power cost from $75 000 to $80 000 each, or about $2 500 per ton of tractive power. Therefore, additional locomotives to 1950 will cost about $2 500 000 000 for combined freight and passenger service.

The small increase in the number of locomotives (Curve *P* (Fig. 8)) is due to the large increase in tractive capacity. The average mileage per year per locomotive (Curve *R* (Fig. 8)) is shown merely to point out the pronounced policy of the railroads to increase locomotive power and train loads and decrease the travel of locomotives. This undoubtedly decreased the operating personnel and probably is partly due to the effect of the full-crew laws on operating expenses.

A careful study of Interstate Commerce Commission reports shows that passenger car-miles are about 13% of the total passenger and freight car-miles; that passenger revenue is 19.7% of combined passenger and freight revenue; and that passenger car-miles per locomotive-mile are about 14% of total car-miles per locomotive-mile. This is assumed as justification for

---

[4] Rept., Interstate Commerce Comm., 1924, p. XCVI.

[5] *Loc. cit.*, p. XCII.

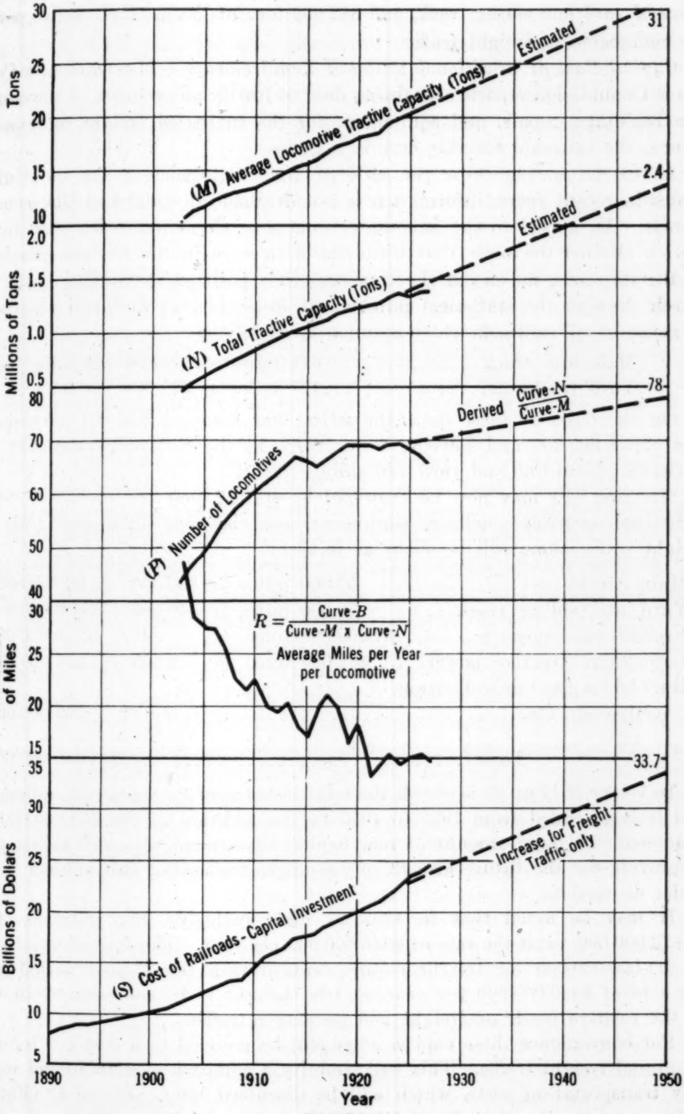

Tons
30
25
20
15
10
Millions of Tons
2.0
1.5
1.0
0.5
80
70
60
50
40
Thousands of Miles
30
25
20
15
Billions of Dollars
35
30
25
20
15
10
5
1890
1900
1910
1920
1930
1940
1950
Year
(M) Average Locomotive Tractive Capacity (Tons)
Estimated
31
(N) Total Tractive Capacity (Tons)
Estimated
2.4
(P) Number of Locomotives
Derived $\frac{\text{Curve-}N}{\text{Curve-}M}$
78
$R = \frac{\text{Curve-}B}{\text{Curve-}M \times \text{Curve-}N}$
Average Miles per Year per Locomotive
(S) Cost of Railroads - Capital Investment
Increase for Freight Traffic only
33.7

FIG. 8.

estimating that about 85% of the additional track and locomotive requirements indicated by Figs. 5 to 8, or, say, 50 000 miles of main track, 70 000 miles of yard and siding track, and 862 000 tons of locomotive tractive power, are chargeable to freight traffic.

Capital Cost of Additional Railroad Requirements.—The Interstate Commerce Commission reports contain no data to justify an estimate of the cost of new buildings, repair, and equipment, for the increased freight movements. Hence, the estimate for this cost is a guess.

As to the average cost per mile of track construction for the United States in recent years, information is not available in published Government reports. An appeal to the Interstate Commerce Commission for such information elicited the reply that while the data were in the records submitted by the railroads, no one at the time was in a position to tabulate them. A search through the statistical numbers of *Railroad Age* disclosed reports of 43 items by 27 railroads which summarize as follows:

Main-line track	612 miles	at $76 000 per mile
Yard tracks	177 miles	at $77 000 per mile

On the basis of these data, the writer has assumed for the purposes of this paper $60 000 and $40 000, respectively, as the cost of construction for additional main-line and yard and siding tracks.

Question (*b*) may now be answered by stating that the probable cost of additional railroad trackage, equipment, etc., between 1925 and 1950 for freight traffic alone will be about as follows:

Main tracks	50 000 miles	@	$60 000	$3 000 000 000
Yard and siding track	70 000 miles	@	40 000	2 800 000 000
Freight car capacity	57 000 000 tons	@	40	2 280 000 000
Locomotive tractive power	862 000 tons	@	2 500	2 155 000 000
Buildings, miscellaneous equipment, etc.				765 000 000
Total				$11 000 000 000

In Curve *S* (Fig. 8) is shown the total investment in roads and equipment, and it is extended from 1925 to 1950 by the addition of the $11 000 000 000 estimated. A large amount of new capital investment also will be required to provide for the future growth of passenger traffic, but this subject is not under discussion.

It may be noted that in 1922 to 1927, inclusive, the railroads spent $3 873 000 000, or at the rate of $645 000 000 per year. The foregoing estimate of $11 000 000 000 for freight requirements during the 25-year period is at the rate of $440 000 000 per year, and is thought to be conservative in view of the relative needs of freight and passenger traffic.

For convenience these capital costs will be reduced to a cost per ton-mile of annual freight traffic. This will simplify a comparison with inland waterway transportation costs, which will be discussed later. Curve *C* (Fig. 5) indicates a density of traffic of 1 600 000 ton-miles by 1950. Hence, the additional 120 000 miles of freight trackage will take care of 192 000 000 000 ton-miles of annual traffic. At a cost of $5 800 000 000, this trackage will average 3.02 cents per ton-mile of freight capacity.

The 57 000 000 tons of additional car capacity (Curve *L* (Fig. 7)) are 35% of the total for 1950 and, therefore, the hauling capacity is 35% of 870 000 000 000, or 305 000 000 000 ton-miles. The cost of this additional car capacity was shown to be $2 280 000 000, which is at the rate of 7.5 mills per ton-mile of annual freight-carrying capacity. The 862 000 tons of additional tractive power are 36% of the total for 1950 and, therefore, the annual hauling capacity is 313 000 000 000 ton-miles, and the additional locomotive cost is 6.9 mills per annual ton-mile capacity. The cost of buildings, etc., distributed over 313 000 000 000 ton-miles amounts to about 2.5 mills per ton-mile of annual traffic.

The sum of these unit costs amounts to 4.7 cents per ton-mile of annual freight capacity. This amount is presented as the probable unit cost of providing additional railroad facilities adequately to handle the increase in freight ton-mileage, which may be expected by 1950.

TRANSPORTATION POSSIBILITIES OF MISSISSIPPI VALLEY WATERWAY SYSTEM

The completion of the projects now (1929) authorized by Congress and by the State of Illinois will provide trunk waterways extending east and west between Pittsburgh and Kansas City; and between New Orleans in the south and St. Paul, Minneapolis, and Chicago in the north. The transportation capacity of these waterways depends principally upon the reliable width and depth of the channel and on the size of the locks. The minimum width of these channels will be 200 ft. The locks on the Ohio River and Illinois Waterway are identical in usable horizontal dimensions, 600 ft. long by 110 ft. wide, and the minimum navigable depth will be 9 ft. The locks on the Upper Mississippi River vary in size and this has been given proper consideration in preparing the statistics of this waterway presented in Table 3.

The estimated capacity by 1950 is materially less than that claimed by the exponents of each waterway and is listed to give a general idea of the transportation possibilities of this waterway system. The total of 60 000 000 000 ton-miles may not be unreasonable in view of the probable growth of transportation in the territory served, provided proper water freight rates should be established. However, such a huge figure should not be considered without, at least, some investigation of the potential tonnage in the area served by these waterways.

According to statistics of the Interstate Commerce Commission report for 1924 the total ton-miles for 30 railroads serving the Mississippi Valley were more than 162 000 000 000. Several of the roads served other territories. After making allowances for these conditions, it was found that about 100 000 000 000 ton-miles might be assumed as within the Mississippi Valley, which comprises 65% of the area of the United States.

Based on the prospective increase indicated by Curve *B* (Fig. 5), an additional 120 000 000 000 ton-miles will materialize by 1950. It does not appear unreasonable to assume that the waterway system could take care of one-third this increase, or 40 000 000 000 ton-miles, equivalent to a water-haul of about 60 000 000 000 ton-miles. This is in view of the fact that the annual traffic of the Federal Barge Line alone on the Mississippi River now (1929) exceeds 1 000 000 000 ton-miles and the uncompleted (1929) Ohio River System produces in excess of 2 000 000 000 ton-miles.

TABLE 3.—MISSISSIPPI VALLEY TRUNK-LINE WATERWAYS.

Stream.	Between.	Length, in miles.	Depth, in feet.	Annual Capacity.		
				Tons.	Billions of Ton-Miles.	
					Maximum.	1950 estimate.
Illinois Waterway......	Chicago and Grafton, Ill............	325	9	60 000 000	20	10
Upper Mississippi River.	St. Paul, Minn., and Grafton, Ill........	670	6	20 000 000	14	4
Mississippi River........	Grafton, Ill., and Missouri River........	20	9	100 000 000	2	1
Missouri River..........	Kansas City, Mo., and Mississippi River..	400	6	30 000 000	12	5
Mississippi River........	Missouri River and Ohio River........	105	9	100 000 000	10	4
Ohio River..............	Pittsburgh, Pa., and Mississippi River..	970	9	60 000 000	52	10
Mississippi River........	Cairo, Ill., to New Orleans, La.......	970	9	150 000 000	100	26
Total..............	....	3 460	....	....	210	60

Assuming 60 000 000 000 ton-miles per annum as the probable traffic demand available for this waterway system by 1950 raises the question as to whether the past growth of water transportation has been such as to indicate a future growth to a total of 60 000 000 000 ton-miles by 1950. The usual reply of waterway advocates to such a question is "provide a dependable completely improved waterway between important terminals and the operating equipment, terminal facilities, and tonnage will be developed." Such a statement is merely a prophecy and without supporting evidence is unconvincing. The writer believes that such a prophecy with regard to the Mississippi River System will eventually come true and that there now exists sufficient evidence in support of such a claim.

The past growth of waterway traffic for the entire Mississippi System cannot be shown, as so much of the system is incomplete and has no traffic. However, the tonnage statistics of all important waterways of the United States are available in public documents, giving the data shown in Table 4. The capacity of the Monongahela River is about 25 000 000 tons, hence, no further material increase may be expected until additional locks are provided.

The other waterways in Table 4 show an annual traffic increase of from three to five times the rate of increase shown by railroads. Such increases are remarkable; if they are maintained, the capacities of all these waterways would be reached in less than fifteen years.

The average annual rate of increase in tonnage for these waterways is 14.4 per cent. This rate, if maintained for a period of twenty-five years, would result in a tonnage and probably ton-mileage thirty-six times as great as in 1925. To those who would dispute the assertion that an annual increase of 15% in the next twenty-five years is not only possible but probable, it is pointed out that not one of the rivers in the Mississippi Valley is completely improved for navigation except the Monongahela River and the Mississippi River below Cairo, and even the latter is subject to interruption of traffic for months at a time due to insufficient depth and width of its dredged channels.

TABLE 4.—TRAFFIC ON CERTAIN IMPORTANT UNITED STATES WATERWAYS.

Waterway.	Tonnage, 1919.	Tonnage, 1925.	Annual increase, percentage (100 R).*
Ohio River	5 000 000	15 737 015	21.1
Monongahela River	17 137 501	23 716 121	5.6
Allegheny River	2 058 069	4 744 843	15.0
Mississippi River Division:			
Federal Barge Line	109 218	910 755	42.0
Tennessee River	600 107	1 940 972	21.6
Erie Division, New York Barge Canal	673 000	1 945 466	19.4
Panama Canal	6 981 996	23 958 826	22.8
Total	32 559 891	72 953 998	14.4

* $R = \sqrt[n]{\frac{T}{t}} - 1$, in which, R equals annual rate of increase, n, number of years in period; t, tonnage for 1919; and T, tonnage for 1925. The percentage equals 100 R.

Until these rivers are completely improved, transportation upon them must be considered as in its infancy and, hence, subject to a rate of increase somewhat comparable to that of the early days of the Sault Ste. Marie ("Soo") Locks and the St. Marys Falls Canal in Michigan. As shown by Government reports, the freight passing through the "Soo" Locks in 1856 was 33 817 tons and, in 1880, it was 1 321 906 tons. This 25-year period shows 15.7% annual increase. In 1905, the tonnage was 44 270 680, equivalent to an annual increase of 15.1 per cent.

The "Soo" Locks are the bottleneck of the greatest inland waterway system of the world. They are frequently cited as an example so favored by Nature that no other waterway could be comparable with them as to tonnage possibilities. Yet neither for the first fifty years of their history, nor for any 6-year period subsequent to 1890, was the annual rate of tonnage increase as great as the rate for the waterways listed in Table 4, the Allegheny and Monongahela Rivers excepted. This certainly indicates that, since 1919, transportation interests, exclusive of governmental, have been utilizing the principal unconnected and incomplete waterways of the Mississippi River System as fully as could be expected. Hence, the claim of opponents to waterway development that the public would not use them is without justification.

Those who so severely attack the policy of improving American waterways as uneconomical dwell at great length on the small volume of tonnage on certain of the waterways listed in Table 4. Such individuals lose sight of or ignore the power of compound interest.

For many years the New York Barge Canal has been a conspicuous and favorite object of attack, yet the main division of this canal, inadequate in size for modern transportation methods, shows an annual tonnage increase of 19.4 per cent. The Erie Division of the New York Barge Canal was designed for a maximum of 20 000 000 tons. The tonnage in 1925 was more than 1 900 000; hence, if the tonnage continues to increase for 25 years at one-half the past rate of increase, or, say, 10%, the capacity of the canal will be reached by 1950.

The waterways of Table 4 show an average rate of increase in tonnage of 14.4 per cent. If the Monongahela River is excluded, the rate is 21.4 per cent. On Fig. 9 are shown graphically these annual tonnages. From 1925 the curves are extended to indicate prospective tonnage based on an annual increase of 10 per cent.

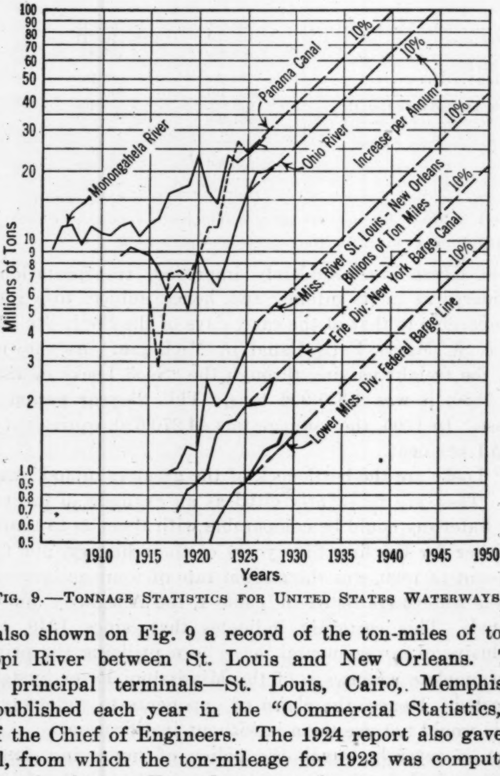


FIG. 9.—TONNAGE STATISTICS FOR UNITED STATES WATERWAYS.

There is also shown on Fig. 9 a record of the ton-miles of total traffic on the Mississippi River between St. Louis and New Orleans. The tonnage between the principal terminals—St. Louis, Cairo, Memphis, and New Orleans—is published each year in the "Commercial Statistics," volume of the reports of the Chief of Engineers. The 1924 report also gave the average haul in detail, from which the ton-mileage for 1923 was computed and it is, therefore, correctly shown. For other years the tonnage was combined with the average haul of 1923 to obtain the ton-miles as plotted. The error is probably small and on the conservative side as it is known that the increase in tonnage is largely on longer haul freight. Fig. 9 shows an annual increase of 18% between 1917 and 1926 and 28% for the years 1924 to 1926. If this particular curve is continued at a 10% rate, it reaches a probable ton-mileage of 44 000 000 000 by 1950.

It would seem that the requirements of conservatism would be sufficiently satisfied if 10% be taken as the annual rate on which to base the future growth of transportation on the Mississippi River System. This rate will produce an eleven-fold increase in 25 years.

Government reports show that in 1925 the Ohio River System, exclusive of the Monongahela, carried more than 1 000 000 000 ton-miles. At 10% annual increase this would amount to 11 000 000 000 ton-miles for 1950.

Thus, the Mississippi below St. Louis and the Ohio below Pittsburgh should supply 55 000 000 000 ton-miles in 1950. It is reasonable to expect that new traffic on the Upper Mississippi and the Missouri augmented by that resulting from opening the Illinois Waterway about 1931, will produce enough traffic to increase the total well in excess of the 60 000 000 000 ton-miles by 1950 estimated in Table 3. On the basis of the foregoing discussion, the writer answers Question *(c)* by stating that the progress in waterway transportation from 1919 to 1925 justifies the assertion that by 1950 the inland waterways of the Mississippi and its tributaries can supply an equivalent amount, or 40 000 000 000 ton-miles of the expected railroad transportation demand of the Mississippi Valley.

COST OF WATERWAY TRANSPORTATION FACILITIES

The costs of providing trackage and operating equipment for railroad transportation were based entirely upon the records of the Interstate Commerce Commission to eliminate as far as possible personal opinions. The following derivation of waterway costs, whenever possible, will also be based on Government data.

The best data on inland waterway costs are the annual reports of the Federal-operated Inland Coastwise Waterways Service which, in 1924, became the Inland Waterways Corporation. This Corporation and its predecessor have operated barge lines on the Warrior and Mississippi Rivers since 1919. The Warrior Division is still in the experimental stage and its operations are unprofitable.

The Mississippi Division hereinafter referred to as the Barge Line has shown steady and persistent progress whenever a proper navigable channel was available. The great variety of general freight carried by the Barge Line and its field of operations in the heart of the Mississippi Valley make it an ideal example for comparing inland waterways and railroads.

The Barge Line in 1925 transported 910 000 tons an average distance of 960 miles, or a total of 875 000 000 ton-miles. In 1925 its total investment in real property and equiment was $8 079 833.43, or at the rate of 9.24 mills per ton-mile of actual traffic.

This value is not a fair one for two reasons: First, the Barge Line uses municipally owned terminals, the cost of which should be considered; and, second, the line did not operate at its normal capacity because the channel was shallow and narrow for several months.

From data furnished by the Inland Waterways Corporation, Table 5 was prepared.

The writer finds that the 9 towboats are equivalent to 7½ standard towboats, each of which normally tows a cargo of 10 000 tons. These towboats at 20 000 miles per year, and normal cargo, could give 1 500 000 000 ton-miles of service. Actually three of these towboats averaged in excess of 20 000 miles for twelve consecutive months and the entire towboat fleet averaged 19 400 miles for a fiscal year.[6] The towboat capacity in Table 5 was found to be 91% of what the capacity would be if assumed on the basis of actual performance by several towboats for continuous periods in excess of a calendar month, including all delays and lay-overs at port; also the estimate of barge capacity of 1 314 000 000 ton-miles was 69% of the capacity assumed on the basis of actual performance for a calendar month.

TABLE 5.—INLAND WATERWAYS CORPORATION, MISSISSIPPI DIVISION.

Item.	Practical capacity, in ton-miles per annum.	Replacement cost.	Cost per ton-mile, in mills.
U. S. terminals, harbor vessels, etc., municipal terminals used	2 000 000 000	$3 490 000 $2 510 000	3.00
Nine towboats	1 500 000 000	2 810 000	1.87
Fifty-one barges	1 314 000 000	3 933 400	3.00
Three self-propelled barges	142 500 000	840 000	5.90

The Barge Line terminals and equipment are not balanced as to capacity. A balanced system may be as shown in Table 6.

This indicates 8 mills per ton-mile of capacity as the unit cost of providing terminals and operating equipment for a barge system for long hauls and a diversified class of freight, the lowest class being grain.

The unit cost of terminals would be higher for shorter haul business of the same class. However, it may be noted that the barge equipment of the Mississippi Division is of the covered compartment type, costing about $45 per ton of cargo capacity; whereas open barges as used on the rivers for coal, ore, lumber, sand, and gravel, cost less than $25 per ton of cargo capacity.

TABLE 6.

Item.	Normal capacity, in ton-miles.	Unit cost, in mills.	Total cost.
Terminals and harbor equipment	1 000 000 000	3.00	$3 000 000
Towboats	900 000 000	1.9	1 710 000
Barges	900 000 000	3.0	2 700 000
Self propelled barges	100 000 000	5.9	590 000
Total for System	1 000 000 000	8.0	....
Total cost	....	....	$8 000 000

To the unit cost of 8 mills for terminals and operating equipment must be added the unit cost for completion of the Mississippi River System as authorized by Congress and the State of Illinois. The U. S. Engineer Department estimates that $125 000 000 will complete all authorized projects in the Mississippi River System. To this should be added $20 000 000 authorized by the State of Illinois to connect the Chicago Sanitary Canal with the Government-operated Illinois River at Utica, Ill., and $5 000 000 or less to provide a 9-ft. open channel in the Illinois River from Utica to Grafton and in the Mississippi from Grafton to Cairo. The total cost for the completion of this Waterway System may be placed at $150 000 000, which for the 60 000 000 000 ton-miles of expected annual traffic previously estimated would amount to 2½ mills per ton-mile.

---

[6] Annual Rept., Inland Waterways Corporation, 1923, p. 52.

Thus, on the basis of the foregoing estimates, the cost of completing approved waterways and providing barge transportation facilities on the main arteries of the Mississippi Valley Waterway System and the Lakes-to-the Gulf Waterway (Question (*d*)) is found to be about 1 cent per ton-mile of annual traffic estimated for 1950.

Costs of Freight Transportation

The railroads being common carriers, the cost to the public of freight transportation by rail is established by published freight rates, as approved by the Interstate Commerce Commission. A different condition exists relative to waterways, due to the fact that the right of way or river channels are public waters which may be freely used by any or all, subject only to navigation regulations established by law. This condition, therefore, permits of three classes of carriers on waterways, namely, common, private, and contract carriers.

Common carriers, such as the Inland Waterways Corporation, have regular routes and schedules; they accept whatever business is offered by the public. Private carriers, on the other hand, are owned and operated solely for the benefit of their owners in transporting their own products. Among existing private carriers may be mentioned several coal companies on the Upper Ohio and Monongahela Rivers, the West Kentucky Coal Company operating on the Tennessee, Ohio, and Mississippi Rivers, the Carnegie and the Jones and Laughlin Steel Companies, and the Standard Oil Company, on the Lower Mississippi River and New York Barge Canals. Contract carriers, many of which have been operating for years on inland rivers, function similarly to the tramp freighters on the oceans.

The water common carrier, to be successful, must have joint rail-and-water rate agreements with the railroads. Hence, these rates are subject to the approval of the Interstate Commerce Commission. The Barge Line is the only water common carrier of consequence in the Mississippi Valley. Its rates, as authorized by the Commission, are made by deducting from the all-rail rate from point of consignment to point of delivery 20% of the all-rail rate from the Barge Lines' port of shipment to port of discharge. For example, at present, a Chicago company desiring to ship agricultural implements to New Orleans can utilize water shipments only between St. Louis and New Orleans, and its saving would be 20% of the $13.00 (St. Louis-New

Orleans rate), or $2.60 per ton. The all-rail Chicago-New Orleans rate is $14.60 per ton; hence, the joint rail-water rate is $14.60 less $2.60, or $12.00 per ton.

In 1926, the Mississippi Division of the Barge Line handled 1 044 649 tons of freight on which the direct freight-bill saving to the shipper averaged $1.75 per ton, or a total saving of $1 828 135. All water carriers are free to set their purely all-water port-to-port rates without reference to the Commission. Hence, it may be expected that all-water rates will be governed by the necessities of competition.

The Barge Line has no competition at present and its "water rates, like all other transportation charges, are in the original instance made to secure as high revenue as the traffic will stand and develop." The high revenue, however, as charged by the Barge Line is 80% of the corresponding all-rail rate and the general shipping public saves the 20 per cent.

The future development of privately owned and operated water carriers must depend upon the profit to be gained in this line of business. The average profit or loss per ton of freight handled by the common carrier is the margin between the average operating cost and revenue. The annual reports of the Inland Waterways Corporation show in great detail the operating costs of the Barge Line, some of which are summarized in Table 7. Maintenance and depreciation is more than 10% of the property investment, which on December 31, 1927 amounted to $10 993 361.23. Neither taxes nor interest are paid as this is a Government-owned corporation.

TABLE 7.—MISSISSIPPI RIVER DIVISION OPERATING COST AND REVENUE PER TON.

Year.	1923.	1924.	1925.	1926.	1927.	1928.	Average.
Terminals:							
Operation	$1.52	$1.36	$1.41	$1.56	$1.42	$1.27	$1.42
Maintenance and depreciation	0.21	0.14	0.18	0.21	0.21	0.17	0.19
Vessels:							
Operation	1.11	0.96	1.01	1.02	1.05	0.84	1.00
Maintenance and depreciation	0.63	0.80	0.64	0.58	0.77	0.60	0.67
Damages	0.08	0.14	0.07	0.09	0.14	0.12	0.11
Traffic, general, and miscellaneous	0.35	0.40	0.29	0.36	0.42	0.37	0.36
Total cost	$3.90	$3.80	$3.60	$3.82	$4.01	$3.37	$3.75
Revenue	3.40	3.64	3.91	4.32	4.22	3.86	3.90
Profit	−0.50	−0.24	0.31	0.50	0.21	0.49	....

As shown by Table 7 the Barge Line has established a cost of operating towboats, barges, and terminals, which information is vitally necessary for prospective future operators of private lines. The average length of haul of the Mississippi Division is slightly less than 1 000 miles. Therefore, Table 7 gives very closely the cost per ton-mile of haul by reading mills instead of dollars for the costs as given.

The minimum, yet very profitable, rate charged by the Barge Line is 2 mills per ton-mile on export grain. The cost of loading and unloading is borne by the consignor. The rate on bauxite ore is about 2½ mills per ton-mile, which includes loading costs only.

In making comparisons between railroads and waterways, the difference in distance *via* rail or water should be considered. For the Mississippi Valley System as a whole 1 mile of rail haul is equivalent to about 1½ miles of water haul. In Table 8 are given typical rail and river distances, together with corresponding rates.

TABLE 8.

New Orleans, La., to:	Distance, in Miles.		Cost per Ton.	
	Rail.	River.	Rail, fifth-class.	All-river.
Cairo, Ill.	554	969	$15.00	$3.90
St. Louis, Mo	716	1 167	17.20	4.24
Chicago, Ill.	921	1 519	19.40	4.84
St. Paul, Minn	1 330	1 850	24.20	5.40
Cincinnati, Ohio	836	1 468	18.80	4.75
Pittsburgh, Pa	1 147	1 937	22.10	5.55

The all-river costs are estimated from the six years, 1923 to 1928, assumed as average revenue per ton between New Orleans and Cairo plus 1.7 mills per ton-mile of additional river distance to cover added cost of vessel operation. About 70% of the total river traffic consisted of grain, sugar, and bauxite ore, hence the river rates offer no basis of comparison with fifth-class rail rates, except to indicate how profitable fifth-class freight would be to water carriers.

It is quite evident that a privately owned line with fixed charges of taxes and interest could not survive as a common carrier under the operating costs and revenues allowed this Federal Barge Line. However, there are two items which give promise of substantial reduction, namely, wharf labor, now costing more than 90 cents, and fuel, 60 cents, per ton of freight handled. The type of equipment is steadily being improved and the effect of this on operating costs is marked. For instance, the cost of operating the Barge Line's tunnel-propeller towboats for 10 months in 1925 was 1.29 mills per ton-mile, whereas that for two stern-wheel towboats was 2.56 mills.

While the profitable operation of common carriers by private capital is yet to be demonstrated, the same is not the case with respect to private or contract carriers. The private carrier usually will be a quantity producer, also, occasionally, a distributor of a more or less specialized product. The investment and operating cost of terminals and floating plant for quantity products will always be materially less than for the common carrier, which at all times must be prepared to handle a great variety of items of commerce in relatively small quantities. If such were not the case, many large corporations would have utilized the Federal Barge Line on the Mississippi River instead of developing their own private lines. Smaller corporations with occasional barge-load shipments will be a prolific source of business for the contract carrier.

At the present stage of the renaissance of inland waterway transportation too little consideration is apt to be given to the contract carrier. A towboat provides a very attractive and comfortable home for an entire family as well as a means of livelihood. There are many such on the rivers to-day, and their number will increase rapidly as commerce develops. In fact, their existence in numbers might prove very embarrassing to the common carriers on account of competitive rate-cutting on port-to-port business.

ECONOMIC ADVANTAGE OF INLAND WATERWAY TRANSPORTATION

The minimum direct benefit of waterway transportation to the shipping public has been fixed by the Interstate Commerce Commission for the Barge Line at 20% of the rail rate from port-to-port of the water haul. It is fair to assume that this will hold for the entire Mississippi Valley System.

The average rail rate of the United States is about 11 mills per ton-mile; hence, the minimum freight-rate saving will be 2.2 mills per ton-mile of rail-haul, or about 1.5 mills per ton-mile of water-haul. On the basis of 60 000 000 000 ton-miles of water-borne traffic by 1950, the total freight rate saving would amount to about $90 000 000 per annum.

Another very substantial economic advantage of waterway transportation is the saving in interest on the difference in capital cost of rail or water-transportation facilities now under discussion. The additional capital requirements for railroad construction as developed in this paper are premised upon an improvement in the efficiency of railroads amounting to about 50% between 1925 and 1950. Then there would be no doubt but that the railroads would earn 5¾% in excess of the stipulated fair rate of return, and hence this rate of interest on capital cost may be assumed.

As previously developed 60 000 000 000 ton-miles of waterway transportation facilities at 1 cent per ton-mile of capacity, would cost $600 000 000, whereas the equivalent railroad capacity of 40 000 000 000 ton-miles at 4.7 cents per ton-mile, would cost $1 880 000 000. The difference in favor of waterways is $1 280 000 000 and the interest charge saving at 5¾% amounts to $73 600 000 per annum.

It may be argued that this interest on saved capital cost may not be set up to the advantage of waterways as an economic saving, on the ground that it is immaterial to the investing public whether the capital is invested in railroads or elsewhere. This may be true so far as the owners of capital are concerned, but it is not true with regard to industry. The interest on capital invested in an industry, where the product can be furnished at the same or less cost by other methods without using the additional capital, is wasted so far as that industry or the users of its product are concerned.

Capital saved from investment in an industry where it is not needed remains available for investment where it is needed, or as part of a capital surplus tending to reduce the interest rates which are a tax on industry and on the economic welfare of the Nation.

Therefore, the writer presents as his answer to Question (*e*) that the minimum economic advantage of inland waterway transportation on the Mississippi Valley System by 1950 will be an annual freight-charge saving of

$90 000 000 plus an interest-charge saving of about $73 600 000, or a total annual saving of about $160 000 000 based on a waterway freight movement of 60 000 000 000 ton-miles per annum.

CONCLUSIONS

The writer's conclusions, as developed in this paper, are as follows:

(*a*-1).—The annual freight transportation demand of the United States by 1950 will be 870 000 000 000 ton-miles, or 470 000 000 000 in excess of 1925.

(*a*-2).—Railroad renewals and increased operating efficiency may provide for an increase of 240 000 000 000 ton-miles.

(*a*-3).—Additional transportation facilities must be provided for about 230 000 000 ton-miles.

(*b*).—Additional railroad facilities for 230 000 000 ton-miles will cost about $11 000 000 000, or 4.7 cents per ton-mile of practical ton-mile capacity.

(*c*-1).—Additional freight demand of the Mississippi Valley by 1950 will be 120 000 000 000 ton-miles, 60 000 000 000 of which will require additional transportation facilities.

(*c*-2).—The Mississippi Valley Inland Waterway System, when completed, will provide an improved right of way sufficient to accommodate more than 200 000 000 000 ton-miles of freight traffic, if it were available.

(*c*-3).—The growth of waterway traffic on the Mississippi System between 1919 and 1928 has been equivalent to more than 15% annual increase. If this is continued at 10% annual increase with new completed waterways added, it will produce 60 000 000 000 ton-miles per annum by 1950, or the equivalent of 40 000 000 000 ton-miles of railroad traffic.

(*d*).—The cost of completing present approved project waterways and providing transportation facilities thereon at 1 cent per ton-mile of traffic expectancy will be about $600 000 000 for 60 000 000 000 ton-miles capacity of floating plant and necessary terminals.

(*e*).—The economic advantage of completing and utilizing the Mississippi Valley Inland Waterway System by 1950 will amount to a minimum of $160 000 000 per annum.

RELATION BETWEEN RAIL AND WATERWAY TRANSPORTATION

By E. A. Hadley,[7] M. Am. Soc. C. E.

Synopsis

The proposition to improve and maintain inland waterways at Government expense, particularly on the larger rivers, leads to the consideration of the economic aspects of such an undertaking which may result in a reduction of earnings to other transportation agencies through Government subsidized competition.

The writer endeavors to set forth the situation in such manner as to point out the inequalities which he believes would result from expenditure of the vast amounts of Government funds which would be required to provide and maintain adequate navigation channels on the principal American rivers to be used in competition with the railroads.

The paper traces the development of transportation in the United States and sets forth the present relation of the railroads to the Government through its regulation of rates and profits from operations.

The result of Government operation of barge lines in the Mississippi-Warrior Service is commented upon and comparison is made of the cost of construction of railroads in place of the proposed improvement of the St. Lawrence Waterway.

The paper closes with an appeal for consideration of the economic phases of expenditure of Government funds to foster private enterprise when such funds are secured largely from taxation of the whole people.

Although this subject is sufficiently broad to include transportation on the Great Lakes as well as coastwise traffic where interchange is had with the railroads, the treatment will be confined largely to the relation between the railroads and the inland waterways, as represented by the larger rivers.

Early Development

In the beginning of the Nation's history waterways were the natural highways. With the invention of the steam engine and its application to boats, their use for transportation purposes became almost universal. The Civil War broke in twain these lines of communication between the North and South and that infant industry, the railroads, ever pushing westward, attracted and took the business, offering facilities cheaper, more convenient, and more expeditious than the rivers.

---

[7] Chf. Engr., Mo. Pac. R. R., St. Louis, Mo.

It was transportation, particularly rail transportation, that speeded up agricultural development. Water transportation, in the early days, made collection and distribution possible, but the great West would have remained unsettled had not steam been found applicable to the purpose of transportation.

In the "good old days" of steamboat transportation the farmer's wheat came from such fields of the Northwestern States as were adjacent to the water routes and was handled down the river by steamer to the great markets of the time—St. Louis, Mo., and New Orleans, La. Examination shows that the rates were much in excess of any current charges.

To permit shipment by boat the farmer was obliged to pack his grain in 2-bushel sacks costing approximately 20 cents each or, say, 10 cents per bushel. Grain is now hauled in bulk by the railroads from the "Twin Cities" to Chicago, Ill. (425 miles) for 8.1 cents per bushel, or less than the cost for sacking alone prior to the Civil War.

To meet the railroad competition it was necessary for the steamboats to adopt more efficient methods and to provide better facilities. Special grain barges came into existence and did away with the necessity of sacking the grain. For many years following the "period of reconstruction" there was probably the best co-ordination of rail and water transportation that the United States has ever experienced.

The necessity for transfer between barges and railroad cars involved expense as well as extensive elevator facilities at both the north and south ends of water routes. As the sources of grain extended back into the interior and the haul by rail increased into the Northern elevators, it was found more expedient and economical to carry the grain the entire distance by rail to the exclusion of the steamboats and barges.

STATUS OF AMERICAN RAILROADS

The United States, with less than 8% of the world's area and less than 6% of the world's population, has more than 85% of the world's railroad mileage. It is generally conceded that American railroads give the best service at the lowest cost; that no other transportation medium can handle the enormous crops or the immense amount of raw materials, as their lines reach to all producing centers and into all markets of the country, both great and small.

The rivers do not produce any new traffic. What they do haul is taken away from the railroads, which, as a whole, are now equipped to handle a greater amount than that represented by their present traffic plus that handled by the barge lines. It is questionable whether inland water transportation on the rivers can be revived and improved to such an extent as to enable it to compete successfully with the railroads of to-day, which are increasing in efficiency from year to year and rendering it correspondingly more difficult for other methods to compete with them.

LIMITATION OF RATES AND PROFITS

The statement often made, that the development of the inland waterways and their operation will lower freight rates, is contrary to the facts. River rates are now placed at a percentage of the railroad rates. Therefore, no incentive exists for the railroads to lower their rates, as the river rates would then be correspondingly lowered and the same percentage of differential would continue to exist.

The Transportation Act of 1920 provides that the railroads shall be permitted to earn a reasonable percentage on the value of their property devoted to transportation purposes. Obviously, any other agency tending to take revenue away from the railroads, whether operated by private capital or subsidized by the Government itself, is not working toward a lowering of rates but, on the contrary, will have a tendency to increase the rates in order to provide gross earnings necessary for the fair return contemplated under the law.

The Government restricts the profits of the railways as it does not those of any other large industry. It subsidizes competition with them as it does not with any other industry, by spending billions of dollars in improving highways and waterways upon which other carriers operate without paying adequately or submitting to such regulation of their service and rates as do the railroads.

COMPETITION WITH SUBSIDIZED WATERWAYS

The territory known as the Mississippi Basin is the chief wealth-producing section of the United States. This area contains 54% of the nation's population and 66% of its rural population. It covers 64% of the total area and produces 70% of the total agricultural products, 52% of the total manufactured products and 68% of the total exportable products. It is in this territory that the Government is carrying forward its "experiment", under the direction of the Inland Waterways Corporation, to demonstrate the supposed economies of a water-transportation system by improvement of the navigable rivers and operation of barge lines thereon.

It is the privilege of the American people to undertake the development of any transportation agency they desire. On the other hand it is contrary to American principles to expend the money of all the people for the development of an enterprise which, due to its location, can be helpful to but a small percentage of the people and even then only by doing great injury to another branch of the same industry upon which all the people must depend.

If river transportation can be produced at a substantially lower cost to the public than railroad transportation, the public would be justified in encouraging such development and using such a method of transportation in place of railroads. As the railroads cannot be discarded they must be supported. Any arrangement entered into by the Government whereby the public pays more for transportation and the individual shipper less is not economically sound.

REAL COST OF TRANSPORTATION

It has been stated that:[8]

"Our Great Lakes carry more freight and at a cheaper rate per ton-mile than any other inland waterway system of the world. The annual savings in transportation costs on the lakes are also greater than the entire cost of all improvements that have been made."

---

[8] In an address by the late Edgar Jadwin, Major-General (*Retired*), U. S. Army, M. Am. Soc. C. E., before a joint meeting of the Society of Terminal Engineers and the New York Post of the Society of American Military Engineers.

Nothing approaching this situation, however, can be said of river transportation as now sponsored by the Federal Government.

In considering the real cost of any form of transportation it is necessary to take into consideration all factors entering into that service, not only those which are positive in character but also those which may be negative, this latter being best exemplified by the item of taxation. The railroads of the United States pay in yearly taxes almost $400 000 000; the Government-operated barge lines pay nothing. The railroads furnish and maintain at their own cost their right of way, roadway structures, and signal systems. All these items are furnished to the barge lines by other departments of the Government without charge.

If all the costs incurred by the Government through its various departments to maintain navigation on rivers were assembled and charged against the cost of operation of the barge lines, the cost per ton of freight transported would be so enormous that its realization would doubtless discourage further development along these lines. Looking at the matter any way, if the railroads were relieved of taxes and the cost of maintenance of roadway and if their terminals were provided at the expense of the large cities, as is now done for the barge line service, doubtless their rates could be reduced far below those now charged by the barge lines.

The rates now assessed for water transportation are certainly not all that is paid through some source for the service which is rendered. The railroads not only lose the traffic handled by the river, but see a part of their annual tax payments go toward paying the cost of supporting their Government-operated competitor.

The real question at issue regarding the proposed further development of inland waterways is whether or not the cost of transportation to the public as a whole will be decreased or whether only individual shippers would benefit by this service, the deficits through operation and the cost of the improvement work to be paid from taxes or buried in the records of other departments of the Government. If the benefit is to accrue to individual shippers only, then it is unfair to the remainder of the people.

Results from Government Barge Line

Since 1920 the Mississippi-Warrior Barge Line, owned and operated by the Government, has carried about 7 775 000 tons of freight at an average annual loss of approximately $440 000, based on its own system of bookkeeping, covering transportation expenses only. The public deserves to have the accounts stated in such manner that it may find out whether the Barge Line shippers are paying the full cost of transporting their shipments or whether they are being subsidized, and to what extent, by the Federal Government at the expense of the people as a whole.

At the time the Inland Waterways Corporation was established the people were told that the sole purpose was to demonstrate that river transportation was a sound investment for private capital. Until this enterprise can be made to show a profit, taking into account capital investment, taxes, and maintenance, and all other items of expense now subsidized by the Government, there can be no hope of interesting private capital in its operation. Another fact to be noted, especially in connection with the present proposals to extend barge service into the Upper Mississippi and its tributaries, is that these waterways are open for navigation only seven or eight months of the year.

Direct expenditures on these natural waterways, including tributaries, have already exceeded $600 000 000. It cannot be considered sound public policy to reduce railroad earnings, impair railroad credit, and render railroad service inadequate in order that a field may be provided for water services, which at best can, and should be, only supplementary to the railroads.

Substituting Railroads for Canals

The reason advanced for the development of the St. Lawrence Waterway is cheaper transportation costs to reach foreign markets, particularly for American wheat. Available American farm lands are approaching maximum production and the population is increasing. It is possible that by the time the waterway could be made ready for service there would be no grain to export. If there were a surplus there is no assurance that it would be located sufficiently close to the Great Lakes territory to enable it to use the St. Lawrence Waterway.

It has been estimated[9] that three double-track, fully equipped railroads could be built from Chicago to Boston, Mass., at a cost of $260 000 000 each, the total cost of the three railroads being about the estimated cost of the St. Lawrence Waterway; and that each of these railroads, operating throughout the year to its theoretical maximum capacity, could handle eight times as much freight as the proposed waterway. In other words, the three railways would cost only slightly more than the proposed waterway and combined could carry about twenty-four times as much freight.

The New York State Barge Canal has cost more than $270 000 000. Prior to its construction it was estimated that it would handle 20 000 000 tons of freight per year, but in its best year (1928) it handled a little more than 3 000 000 tons. Any one of the railroads operating between Buffalo and New York could have handled this entire traffic without disturbance of its normal business. The canal costs the people of New York State nearly $11 000 000 per year so that the taxpayers would have saved approximately $4 500 000 if all the freight had been handled by the railroads with the State paying the charges, and the shippers would have saved about $3 000 000.

Taxes as Affecting Rates

Railroad transportation rates, however, are declining while taxes are increasing. Rates have been lowered until in 1928 they produced an income of $850 000 000 less than if the rates in effect in 1921 had been applicable.

---

[9] "Economic Features of the Proposed St. Lawrence Waterway," Address by Dr. H. G. Moulton, Pres., Brookings Inst., Washington, D. C., before the Traffic Club of Chicago, on March 21, 1929.

The total earnings of the Class I railroads in the first quarter of the calendar year 1929 were about $8 000 000 greater than in the first quarter of 1926.

Their taxes, however, were more than $7 500 000 greater, the net result being that the public took back in increased taxes almost all that it paid in increased earnings for the handling of a larger freight traffic. The net operating income earned represented an increase of about $37 000 000 over the first quarter of 1926, but it was entirely due to reduction in operating expenses that was secured in spite of advances in wages.

In an address[10] in Washington, President Coolidge issued a warning to the American people regarding the menace to National prosperity that is presented by the enormous increases in expenditures made by the Government and the resulting increases in taxes. He stated that the cost of State and local governments increased from $3 900 000 000 in 1921 to $7 931 000 000 in 1927 and that such a heavy drain on the earnings of the people is the greatest menace to the continuance of prosperity.

The National Industrial Conference Board has emphasized[11] that the Nation's total tax bill in 1923, only five years after the World War, was $7 234 000 000, while in 1928, ten years following the close of the war, it was $9 169 000 000, an increase within the five-year period of 25 per cent. Taxes are increasing faster in proportion than either the population or the National income. In 1923 they were less than $65.00 per capita; in 1927, $76.50 per capita; and in 1928 still more. In 1913 taxes were only 6.4% of the National income. In 1923 they were 10%; in 1927, 12%; and in 1928, 12½ per cent.

SUCCESS WITHOUT SUBSIDY?

The railroads as a whole are not opposed to river transportation if it can be successfully conducted without being subsidized. They have co-operated in a constructive manner in the provision of port terminal facilities and in the application of joint rates. They should not be compelled, however, to meet a competition that cannot stand alone. Further expenditures by the Government to extend river transportation will certainly have to come out of revenues received from taxes, which are already burdensome enough.

The present is an era of great and rapid change—of an economic nature—perhaps the greatest the country has ever experienced. It is not confined to any particular section but is nation-wide and varied in character. Many of these changes are occuring faster than corrective formulas can be devised and this warrants unusual and careful consideration of all measures designed for relief.

The people of the United States are optimists, due largely to the fact that for more than 150 years they have had almost continuous growth and prosperity and have developed a sense of confidence and faith that, as a people, they can overcome any critical situation.

---

[10] Address before the Sixteenth Regular Meeting of the Business Organization of the Government, at Memorial Continental Hall, Washington, D. C., on January 23, 1929.

[11] "Cost of Government in the United States, 1926–1927", pub. in 1928, by the National Industrial Conference Board, Inc., pp. 59 and 82.

With growth in size and strength the Nation must use a proportionately greater effort to comprehend the enlarged scope of economic effects, being not too hasty in applying untried or ill-considered methods of either assistance or control. It must not lose sight of the fact that in the creation and distribution of wealth the public must bear the financial burden caused by every error made in the application of economic theories. The welfare of the whole people must be safeguarded by those in charge of public affairs.

DISCUSSION

Lewis C. Sorrell,[12] Esq. (by letter).—The necessity of some intelligent attempt at forecasting long-run traffic developments, where large capital expenditures for transportation are contemplated, is matched by the extreme difficulty of doing so. The construction of a railway or improved waterway involves the investment of funds for many years; and the wisdom of doing this evidently depends upon the probable demand for such an instrumentality. Any economic unit, such as a nation or a region, is confronted with two questions, namely, how much transportation should be provided, and what instrumentalities should be created to furnish the service? Proceeding from a premise of competition, new facilities might be furnished wherever they could provide a service at less cost than existing agencies; and such generally has been the procedure in the past. A disposition exists to-day, however, to proceed from a premise of co-ordination; and this requires consideration of the effect of new facilities upon the pre-existing ones.

Co-ordination requires, in the case of transportation, a determination of the instrumentality best fitted economically to perform the task; or, otherwise stated, an allocation of the entire task of transportation among the several instrumentalities, so that the aggregate cost will be at a minimum. Considered generally, this viewpoint necessitates a calculation of the entire demand for transportation services, and of the cost of meeting that demand through furnishing different types of facilities. Considered at any given moment of time, the question often becomes that of determining how increasing demand may be most economically satisfied. This is likely to be the case with a dynamic economic system, in which population and wants are increasing possibly at varying rates.

Most transportation facilities, such as highways, railways, and improved waterways, involve heavy outlays of fixed capital, which will be "sunk" for many years. Long-time investments necessitate long-run forecasting of the need therefor. Yet the difficulty of such forecasts cannot be ignored; and the longer the period of time, the greater the difficulty of an accurate forecast. Predicting traffic on American railroads for the next three months, or even for the next year, may be done with considerable accuracy; but attempts to state the probable traffic in 1940 or 1950 involve an appraisal of many elements, some of which are probably unknown to-day. The problem is essentially speculative; almost as much so as it was in the days when the transcontinental railroads were projected. Hence, one must approach such a task with two attitudes, namely, the methods used should be carefully scrutinized for possible errors, and the results even then cannot be accepted too tenaciously. One who essays such a task cannot be held to strict proof of the issues, for such an attitude would probably have prevented the consummation of many improvements in transportation.

Mr. Cornish adopts the method of calculating the probable demand for transportation in the United States in 1950, and then considering whether a substantial portion of the increment might not be supplied at lower cost by improving the Mississippi River and its tributaries. Obviously, the two most significant terms of this method are demand and comparative cost. Demand he identifies largely with ton-miles, a unit customarily used to express the quantity of service rendered in carrying freight over the railroads. Two relatively minor limitations of this method may be noted. The unit of demand adopted for this study is customarily used, because it readily submits to statistical methods. Quantities carried may readily be combined with distance, in ton-miles and passenger miles; and records of these quantities are available for many years. This permits calculation of trends and the adoption of forecasting equations. It must not be overlooked, however, that the transportation service, which is the subject of demand, is much more than weight and distance; it is also a matter of total cost, frequency, dependability, in-transit service, reliability, financial responsibility, speed (or minimum over-all time), and legal liability. These qualitative aspects, as yet, have not been compounded into any unit. The caution must be injected, that a study of aggregate demand in terms of ton-miles will hardly afford a basis for determining what the apportionment of future demand is likely to be, as between railways and waterways, when that allocation must depend upon the free choice of shippers.

---

[12] Prof. of Transportation, The School of Commerce and Administration, Univ. of Chicago, Chicago, Ill.

It has been customary, too, in attempting predictions of traffic growth to use railway traffic as evidence of the general development. In the past, when highways and waterways played a less prominent part than they do to-day, probably no large error resulted from the assumption that the general tendency was the same as that manifested in the growth of railway business; but the recent growth of highway and waterway traffic renders railway traffic less accurate as a barometer of total traffic. This consideration, together with the preceding one relating to the unit of demand, does not necessarily invalidate forecasts based upon them; it simply emphasizes the fact that tools heretofore used are somewhat inadequate for the purpose.

More serious is the assumption that underlies the method of measuring future traffic demand. The method used by Mr. Cornish conceives that the growth of demand for railroad transportation is a function of time. Because of the difficulty of forecasting demand, he concludes that the "best solution lies in assuming that the trend of transportation progress in the United States during the past forty years will continue for the next twenty years." A time series of data is charted from 1890 to 1926; and the trend of that series is projected to 1950, at which date it is estimated that American railroads will probably enjoy a freight traffic demand of about 870 000 000 000 ton-miles.

It is instructive to compare these figures with several other attempts at long-range prediction. In 1924 John Balch Blood, Valuation Analyst of the Interstate Commerce Commission, prepared a study and forecast of railroad traffic—passenger and freight. He adopted the premise that such demand is a function of population, rather than of time; and from his study of the trend of population and traffic he concluded[13] "that ton-mileage varies approximately as the third power of population." His forecast provoked much criticism, especially as regards statistical methods, which were thought to yield extravagant results. The Bureau of Statistics of the Interstate Commerce Commission prepared a study, based upon the years 1900-1915 (selected to eliminate the extreme fluctuations of the World War and post-war periods), the results of which were utilized by the Commission in the reduced rate cases of 1922.[14] In 1923, the Chamber of Commerce of the United States made a report on the "Probable Freight and Passenger Traffic of the Railways of the United States of 1933". Of course these different attempts at forecasting used somewhat different methods and different terminal years. In 1924, L. A. Abbott, a Statistical Analyst of the Interstate Commerce Commission, discussed the methods used in the several estimates, and supplemented them with an independent estimate of his own, bringing them all down to the same terminal year 1943.[15] The several forecasts yielded the following results:

---

[13] *Railway Age*, Vol. 76 (1924), pp. 369–371.

Forecast for the year 1943 made by:	Billions of ton-miles.
Interstate Commerce Commission	494
U. S. Chamber of Commerce	727
L. A. Abbott	623
J. B. Blood	1 167

So far as concerns the demand for railway transportation, it is unsafe to assume that the trend for the next twenty-five years will be the same as that for the last forty. A careful study of the growth of freight traffic on American railroads shows that a noticeable change in the rate of growth occurred about 1910. Dr. M. O. Lorenz, Director of Statistics of the Interstate Commerce Commission, recently made this statement:[16]

"From 1900 to 1914, freight ton-miles approximately doubled, while the ton-miles of 1928 were only about 50% greater than those of the fiscal year 1914. Furthermore, the passenger traffic and tons of freight have failed to maintain the same absolute amount of increase per annum in the second as compared with the first half of the period 1900-1928. Ton-miles of freight, however, show about the same annual addition in absolute amount from 1900-1914 (fiscal years) as from 1914 (fiscal) to the calendar year 1928, the annual addition having been 10.5 billions in the first period, and in the second 10.2 billions. It is not safe to predict the future solely by the past, because new forces come into action. The development of the motor vehicle was such a force, the effect of which on railroad traffic was not fully appreciated in 1915."

Generally speaking, it would seem as though methods of forecasting the trend of traffic, which are based upon the ratio of demand to population, would be somewhat more valid than those relying upon a time series alone. Mr. Abbott questions this,[17] holding that comparison of the different forecasts suggests that those using population growth as a basis have introduced an unnecessary element. It may be said that the time series does include all existing variables, without assigning weight to any single one. The writer has developed two equations of per capita ton-miles, using 1899-1926 and

---

[14] Interstate Commerce Comm. Repts., Vol. 68, p. 701.

[15] *Railway Age*, Vol. 77, pp. 243–247 (1924).

[16] As reported in the *United States Daily*, July 15, 1929.

[17] *Railway Age*, Vol. 77 (1924), pp. 243–247.

1910-1926 as periods for the calculation of the trend. Linear equations were used in the following form:

(*a*) Fiscal Years, 1899-1926; Centered, 1912-1913:

$$Y = 2\,849.890 + 36.698X \qquad (1)$$

(*b*) Fiscal Years, 1910-1926; Centered, 1918:

$$Y = 3\,227.0156 + 50.4473X \qquad (2)$$

in which, Y is the traffic, in ton-miles per capita; and X is the ratio of demand to population.

Equation (1) gives 4 207 ton-miles per capita, and Equation (2) 4 841 ton-miles per capita, as the expected demand in 1950. Mr. Raymond Pearl gives[18] the expected population of Continental United States at that date as 148 678 000. Applying this estimate to that of expected ton-miles per capita, gives 623 000 000 000 and 716 000 000 000, respectively. Most estimates, except those of Blood, would probably assign a considerably smaller value to the expected ton-miles for 1950 than the figure given by Mr. Cornish.

It is obvious that the actual demand for transportation is a function of many variables. Increasing population, together with an improving standard of living, would ordinarily tend to increase the demand for transportation, as expressed in ton-miles. There are numerous forces, however, that work in the opposite direction. Increasing decentralization of industry, substitution of transmission of energy for transportation of coal, reduction in the amount of coal or other raw material required in production, substitution of materials involving short hauls for those requiring long hauls, and importation of commodities formerly produced at home, are a few instances of factors that must be considered in estimating the probable future traffic on inland transportation facilities.

Cost of Transportation.—If the calculation of probable future demand is beset with difficulties, the computation of comparative cost is even more intricate. This is so, not only because cost data are so difficult to secure, but also because agreement may not exist concerning the items that should properly be included. That cost varies with the purpose to be served, and the problem to be solved is well recognized. Where, as with transportation instrumentalities, a large volume of overhead costs exists, questions of policy concerning the apportionment of such overhead may complicate the solution. Likewise, joint expenditures, as for navigation and flood protection, may raise the issue concerning the proper sum to be charged against commerce. Where a long period of time is required to construct the enterprise, interest and operation deficits during such construction period must be considered. Similarly, the period of time required to develop traffic to the point, where the facility enjoys a degree of utilization necessary to realize the economies of its use, may accumulate a considerable burden of "developmental expenses", sometimes capitalized under the expression, "good-will".

The instant problem involves a choice between two alternative methods of meeting an anticipated increase in the demand. Part of this assumed increase may be accommodated by an improved waterway, or by increasing the rail capacity. The choice of alternatives involves an approximation of the total economic outlay required for one instrumentality, as compared with that required for the other.

---

[18] "Studies in Human Biology," p. 590.

The method used by Mr. Cornish is reduced to five logical steps: First, he ascertains the probable increase in railroad investment necessary to afford the requisite increase in capacity. Second, he makes a similar estimate for the waterway. These two estimates are reduced to capital costs per ton-mile of capacity. Third, although not expressly so stated, he evidently regards rail operating costs as the present average per ton-mile earnings; this, of course, includes return on investment and taxes. Fourth, he takes the present earnings of the Inland Waterways Corporation as evidence of carrier-operating costs on the waterway. Fifth, he computes the saving to the shipper under the present 20% differential in water rates under the rail, on the assumed tonnage likely to be realized by the waterway, and adds thereto interest on the difference in capital outlay for waterway and railway, respectively. Limitations of space permit only cursory examination.

Railroad Capital Outlays.—Without attempting a detailed check on the trends in the capital outlays for the several kinds of railroad property, and without essaying the difficult task of estimating the amount of unused capacity now existing in the railroad plant, it may be said that an estimate of an annual investment of about $400 000 000 per year, to provide better freight-carrying facilities, would probably fall within any competent railroad opinion. The exact amount of such outlays will depend not only on the needs of increasing traffic, but also on the desire to reduce costs by substituting capital for labor, and on the necessity of complying with public demand for terminal improvement and grade separations.

Waterway Capital Outlays.—Without checking the estimates of equipment and channel improvement costs, necessary to carry the estimated traffic increase, it may be observed that Mr. Cornish disregards capital outlays already made for river improvement and terminals, except the terminals used by the Federal Barge Line. Doubtless, it is difficult to estimate them, because, in part, they are joint with the cost of flood protection. Where a plant has been built, such that its productive capacity greatly exceeds the market, it may be a justifiable procedure to treat a considerable part of that capital outlay as a sunk cost; "a matter of historical interest but not affected by any subsequent acts of the management".[19] The premise of this argument is not a transportation agency larger than needed, but rather one to be enlarged to meet the demand. The outlays already made for the canalization of the Mississippi will be used to carry that increased traffic, as well as the future capital investment; and it would seem that the annual cost on account of total capital invested should be prorated against the total traffic anticipated in 1950.

Railway Operating Costs.—These, of course, are not identical with average freight receipts per ton-mile; but for purposes of rough comparison that is not very material. Mr. Cornish considers that the present average receipts of 11 mills per ton-mile are not fully compensatory to railroad capital; and assumes that increasing density will probably render the present freight rate level fully so. If railroads now existing in the United States can achieve a density of 1 600 000 ton-miles per mile of all tracks by 1950, assuming no other changes in cost, it is likely that a substantial reduction in rates could take place. This would lessen the savings due to waterway construction.

---

[19] "Economics of Overhead Costs," by Clark, p. 180.

Waterway Operating Costs.—In this calculation Mr. Cornish takes the present ton-mile earnings of the Inland Waterways Corporation as evidence of carrier cost when using this agency. These figures do not include taxes and interest, which a private carrier must meet; possibly, he assumes that expected improvements will take care of these. In order, however, to present figures comparable with the rail carrier's earnings, the taxpayer's contribution should be considered. This would require something added for channel and terminal maintenance and operation, interest on the investment, and taxes foregone by the public on capital thus invested. With the present utilization of that waterway, the inclusion of such costs would probably produce a somewhat unfavorable result; but with the assumption of a much greater utilization in 1950, the added cost per ton-mile because of such overhead costs would be much smaller.

The Economic Benefits of the Waterway.—These are calculated by applying the present freight rate differential to the expected traffic in 1950, and adding thereto interest saved by reason of the much smaller investment required to carry 40 000 000 000 railway ton-miles by water than would be needed to carry that traffic by rail. The estimate of an annual saving of about $160 000 000 would not be impossible of attainment, provided the assumed traffic density on the waterway is actually achieved; but the method of demonstrating this seems confused. The present differential of 20% is not based upon differences in operating costs; supposedly, it is based upon the concession to the shipper necessary to induce him to use the waterway in preference to the railway. That is, it is more nearly a measure of the difference in the valuation of the two services; and, at present, that saving to the shipper is offset in part at least by the taxpayer's contribution in the shape of non-taxable and non-interest earning capital. The saving is not a net social gain; but even supposing that this differential by 1950 did come to represent a net social gain, one would question the accuracy of adding interest saved to savings from freight rates in order to measure the total benefits. This is so because the freight rate differential then would be due largely to savings attributable to a smaller capital outlay for the waterway than for the railway.

A clearer procedure would be to ascertain the probable operating costs of the waterway carriers, not including returns on capital in the waterway itself, and to reduce these to a ton-mile basis. Next, calculate the capital investment in channel and terminals necessary to carry the assumed traffic. Compute the maintenance and channel operating costs, interest costs, and taxes that might be assessed against such a waterway itself; reduce these overhead costs to a ton-mile basis. Add this ton-mile toll to the ton-mile operating costs and thus arrive at the probable social cost of a ton-mile of water transportation in 1950 with an assumed utilization of 60 000 000 000 ton-miles per annum. For purposes of comparison equate waterway and railway ton-miles on the basis of 1½ to 1. The difference between this figure and the average probable railway freight receipts per ton-mile with the density premised for 1950, represents a net social saving due to the use of the waterway instead of the railway.

The remaining issues must be commented upon briefly. Although the expression, "efficiency", commonly means the ratio of output to input in terms of work, and may possibly be used to compare use with capacity to render service, nevertheless it seems preferable to adopt the term, "utilization", for the latter concept, rather than the over-worked term, "efficiency". In calculating the probable growth of waterway traffic on the Mississippi System, it hardly seems wise to include the Panama Canal; as in the comparable case of railway location, traffic studies should be confined to agencies the circumstances of which are not too dissimilar. Should the estimated traffic fail to materialize, the burden of overhead costs per unit of waterway traffic would be considerably increased, and the savings, of course, correspondingly diminished. The same result will follow if the estimate of capital expenditures yet required is considerably below the actual sum needed.

This is in no sense captious criticism. Mr. Cornish has done good service in attempting to give a quantitative expression to the variables in this problem. Transportation must be studied comparatively, with reference both to the service and the cost. Economic, engineering, and accounting methods must all be utilized in the process. The subject bristles with difficulties; and early attempts at solution will of course involve errors. It may not be too much to hope that further studies of this kind will follow; and that as a result something like a general acceptance of general principles may accrue, and that methods of solving these problems of transportation co-ordination may steadily improve.

R. B. KITTREDGE,[20] M. AM. SOC. C. E. (by letter).—At this time, when extravagant claims both for and against inland waterways are being so freely made from numberless platforms, and so widely discussed in the public press, there rise in many minds questions of reasonable doubt as to whether the further development of inland waterways would result in a waste of the taxpayers' money or in a substantial means of relief to a distressed Middle West and to a depressed agricultural industry. The paper by Mr. Cornish presents the opportunity for thoughtful discussion by the profession, which should do much to clarify the situation.

One of the conclusions of this paper is that, after the completion of waterways now authorized, the Mississippi Valley Inland Waterway System by 1950 could produce annually the equivalent of 40 000 000 000 ton-miles of railway transportation at a minimum economic advantage of $160 000 000 per annum. If true, this is important.

Apparently, however, the conclusion has been drawn as the result of an analysis which includes two errors of major importance: First, figures which cover the total cost of railway transportation have been used in comparison with other figures which include only a part of the cost of waterway transportation; and, second, the railways have in effect been charged not only once but twice with the interest on a substantial part of their investment. When these interest charges are estimated correctly, and when the costs compared are the total costs, the logical conclusions as to the economic advantage of completing this waterway system may of necessity be materially modified.

---

[20] Prof. of Transportation Eng., State Univ. of Iowa, Iowa City, Iowa.

Mr. Cornish has estimated at $1 880 000 000 the investment required to expand railway facilities for an additional 40 000 000 000 ton-miles of railway traffic. He has also estimated at $600 000 000 the investment necessary to supply an equivalent amount of waterway transportation by the completion of certain partly constructed waterways in the Mississippi Valley. Capital to the amount of $1 280 000 000 is thus assumed to be saved for investment elsewhere if, by the completion of these waterways, the indicated expansion in railway facilities is indeed avoided. At this point, by adding to an estimated $90 000 000 annual saving in freight charges the sum of $73 600 000 to cover the annual interest, at 5¾%, on this additional investment of $1 280 000 000, Mr. Cornish arrives at a figure of $160 000 000—this representing, in round numbers, the minimum yearly economic advantage of waterway transportation on this Mississippi Valley System. In this computation consideration has nowhere been given to a considerable part of the cost of waterway transportation, and interest charges have been handled incorrectly.

The author's $90 000 000 of annual saving in freight charges was estimated by using rail rates of 11 mills per ton-mile, and water rates of 80% of the equivalent rail rates. The rail rates thus used would cover the total cost of the rail transportation, including taxes, all railway maintenance costs, and interest on the total railway investment. On this waterway system, rates fixed at 80% of the equivalent rail rates could not be expected to cover the total cost of the waterway transportation, including an allowance for taxes, the maintenance costs of the waterways, and interest on the additional investment required to complete their construction.

For some time past, the average railway freight rate has been approximately 11 mills per ton-mile, and railway revenues over this period have covered the total cost of railway transportation, including taxes, and all maintenance costs, and whatever interest has been paid on railway investment. On inland rivers and canals, with the exception of a part of the Ohio River System and of the connecting channels of the Great Lakes, water rates at 80% of the equivalent rail rates have not been sufficient to cover the total cost of the waterway transportation. The basis for such rates is what the traffic will bear and is not what the transportation by water actually costs. Low rates on inland rivers and canals have been possible only because the taxpayers have constructed and maintained the waterways and have also waived taxes and interest upon them. If the railways were similarly relieved of maintenance, taxes, and interest, railway rates might well be cut in two. This does not mean that transportation development on inland rivers is never justifiable, but it does mean that, before its economic justification can be established, full consideration must be given to its total cost.

When waterways are constructed and maintained at governmental expense, the saving to the shippers involves a sacrifice by the taxpayers which by no means should be overlooked. Maintenance costs of the waterways, interest on their construction cost, and taxes not assessed, are all matters of more than academic interest and, because this is true, they all deserve careful consideration. Although a part of the cost of waterway transportation may be concealed for a time, in the end it cannot be escaped. The burdens on the taxpayers may be less in evidence than are the savings to the shippers, but, by virtue of their being less apparent, they are in no sense less real.

Instead of deducting from the $90 000 000 estimated annual saving in freight charges the sacrifices required of the taxpayers to make this saving possible, Mr. Cornish has added $73 600 000 as representing 5¾% interest on $1 280 000 000 of the railway investment. This interest has been added in spite of the fact that, when railway rates were estimated at 11 mills per ton-mile, interest on all the railway investment, including this $1 280 000 000, had already been charged once against the railways. Mr. Cornish clearly estimated that railway rates of 11 mills per ton-mile would be sufficient to cover interest on all the railway investment. Interest on a part of this investment should not be charged against the railways again.

If, by constructing waterways, $1 280 000 000 of capital, which would otherwise be invested in railway facilities and earning 5¾% interest, could indeed be released for investment elsewhere, some benefit would undoubtedly result to the credit of the waterways, although such a benefit would not amount to any substantial part of $73 600 000 annually. The benefit of releasing for investment elsewhere capital already earning 5¾% interest would not amount annually to another 5¾% of the capital.

Wherever the saving in freight charges is less than the sacrifice required of the taxpayers, the construction and operation of inland waterways would result in an increase in the total cost of transportation. In the face of this condition, the economic justification of the waterways must be based squarely on some benefit in addition to the saving effected in freight charges.

Other benefits, in addition to the savings in freight charges, and at the same time certain other disadvantages, in addition to the contributions by the taxpayers, would follow the development of an inland waterway system. All these, of course, should be carefully evaluated before drawing any final conclusion as to the economic justification of waterway transportation.

Mr. Cornish has clearly formulated certain definite questions with reference to railway and waterway transportation, and he has courageously given both his own answers to these questions and the process of reasoning which lay behind them. Inevitably some errors will creep into so comprehensive a discussion; and the writer believes that errors have been made which result in over-estimating the value of waterway transportation. Errors which may exist in the paper, however, should not be allowed to obscure all of value which there remains.

The completion of this Mississippi Valley Inland Waterway System may be justifiable; the advantage, however, from a purely economic standpoint, of its immediate completion and use has not to the present time been convincingly demonstrated.

Henry A. Palmer,[21] Esq. (by letter).—An understanding of a few fundamentals should precede any consideration of inland waterway development, or the lines along which such development should proceed. Some of them do not seem to be understood generally.

The movement for the development of inland waterways as an agency for the transportation of freight is due to (1) the passion of the engineer for the utilization of natural waterways; (2) the desire of certain localities and business interests for lower freight rates—no matter at whose expense; and (3) the willingness of demagogic politicians always to be "for" what they think their constituents desire.

Any policy of this sort, however, should be based on something more sound. There is nothing sacred about any kind of transportation and, if it is found that water transportation fills a need that can be supplied in accordance with sound and reasonable economy, it should, of course, be supplied. Indeed, it will be supplied, because capital is constantly on the lookout for such opportunities; but it ought not to be supplied under any form of paternalism or conditions that require support to be furnished by the public in order that those who benefit may prosper.

Thus, wherever there is, on adequate and fair investigation, found to be a reasonable demand for the development of a waterway and a reasonable assurance that traffic will seek it, once it is developed, the Government should undertake to develop that waterway. The development, however, ought to be with the understanding that operators of transportation on it shall be subjected to the same sort of regulation that governs their competitor, the railroad; that rates shall be based on a real cost study, which has never been made; and that operators shall pay into the public treasury a reasonable fee for permission to use the waterway as a place of doing business for profit.

Under present conditions inland waterway operators are permitted to make rates arbitrarily 20% less than rail rates. There is no economic basis for that differential; that is, no one knows just how much less the cost of water transportation is than that of rail transportation, if any. Waterway carriers make their own regulations and do things for shippers that the railroad companies, under strict regulation, are not permitted to do.

Furthermore, whatever else may be permitted, the Government should retire from the transportation business of the Mississippi and Warrior Rivers. Government in business of any sort is not in accord with approved public policy. In this particular case it causes sad complications because the Government corporation that operates this business insists on showing a profit, and that profit is computed with no overhead expense, such as a private operator must set up—no taxes, no insurance, and no cost of capital. For that reason, it is not a fair exhibit of what can be done in waterway transportation, although Government operation on these waterways purports to be an experiment for the purpose of demonstrating to private capital what can be done. Of course, private capital will not be fooled, but the public is fooled and has an extremely false idea of the cheapness of water, as compared with rail, transportation. This confused idea prevails even in Congress.

---

[21] Editor and Mgr., *The Traffic World*, Chicago, Ill.

Much is said in favor of developing waterway transportation on the theory that the railroad companies may not be able to take care of the country's growing traffic. There can be little to this if the Government carries out the spirit and even the letter of the Transportation Act of 1920, which is that the railroads shall be enabled to earn a fair return on the money invested in them. If they are permitted to do this they can be counted on, in their own interest, to make the extensions necessary to care for increasing business.

Admit for the sake of the argument, however, that the railroads may not be able to cope with increasing traffic demands, as time goes on, either because they are incompetent or because they are so restricted in earnings as not to be able to do the things that ought to be done and that they would like to do. Inland waterway development might then become advisable; but why develop the waterways under unfair conditions, even then? Why not place them on a sound economic basis at once, instead of providing here and there facilities for those who can use them at the expense, in part, of the people as a whole? Why not make those who use them pay for what they get? Certainly, no one in his senses has ever advocated such a system with respect to rail transportation; but, when the subject is carefully thought out, if the Government concludes that it must provide cheaper and more transportation facilities, why should it not build a railroad, for instance, between Chicago, Ill., and New Orleans, La., charge rates 20% less than the rates it permits privately owned railroads to charge, pay the deficit out of the public treasury, and then tell the people not only that it is giving them low rates, but that the operation is profitable? Any one capable of thinking can see that such a thing would be unfair to the railroad companies, subversive of sound transportation policy, and indefensible in economics. Yet what is the difference, in principle, between such a course and the one that is being pursued?

Selfish and unsound views with respect to this question may be expected from those who have interests at stake—the railroad companies, on the one hand, who do not want this competition, and shippers, on the other hand, who can use waterways and can profit by the low rates offered. Engineers, economists, and statesmen, on the other hand, ought to clarify their views and act in accordance with common honesty and sound practice.

J. PARKER SNOW,[22] M. AM. SOC. C. E. (by letter).—The Symposium sets forth an imposing array of data on various types of transportation as well as many valuable comments on the ethics of Government improvement of public ways by land and water. The discussion here concerns the paths or ways of transportation rather than the substance carried or the commercial aspects of the great subject; and railroads and waterways are the special ways in question. Passenger traffic on these ways seems to be a decreasing factor at present, and freight traffic only will be discussed herein.

---

[22] Cons. Engr., Boston, Mass.

To treat the subject rationally it must be recognized that there are two fundamentally different classes into which these ways naturally divide. Railroads are economically essential monopolies, privately owned and operated, while waterways are of two kinds. Artificial canals, whether owned privately or by the Government, are properly, like railroads, essential monopolies; while the sea, lakes, and navigable streams are, like highways, properly free for all who furnish the vehicles which they use. Ways of the first kind are common carriers and should be under public regulation. They are commercial ventures; no one but their owners can use them, and the traffic they carry must pay rates for the service. On the other hand highways and natural waterways are open to free competition to all proper use. They are not, speaking broadly, well adapted for use by common carriers. General Ashburn intimates this as applying to waterways. His Table 1 shows a slight net income in common carrier service on waterways for 1928; but, if a fair return on cost of equipment and for depreciation had been included, the net would have been in red type. If freight rates were computed on a rational basis, a just comparison of them could be made which is impossible under the present American method. The German method for a generation or more has been to fix the rate on the sum of the costs at each end of the run and that of the line haul. If this method were in use in the United States much fairer comparison could be made than with the rates now existing. Street-car lines occupy highways and are common carriers, but they furnish their own track and vehicles and are necessarily in the first classification. Even buses must be so classed if used as constituent parts of car lines or railroads, but sound judgment is needed in granting or denying permits for competing bus lines.

Ways of the second class must be adapted and maintained by the public. Municipalities, with assistance from the State and Nation, handle highways, while the Federal Government maintains the waterways. Proper maintenance means improvement to keep step with advancing means of transport. The sums spent to make and keep harbors and navigable rivers properly usable seem enormous, but they are small compared with the vast aggregate sum spent on highways. The cost of improving highways can happily be kept reasonably low to the common taxpayer by means of the gasoline excise. Improvement of natural waterways must be done by the Government; it is not subsidy; it is justifiable improvement, and there seems to be no way to pay for it except by appropriations from the Treasury. Unhappily, the securing of such appropriations sometimes leads to "log-rolling bees" in Congress. Whatever the difficulties may be, however, public ways must be kept abreast with the march of progress.

Railroads are old; vast progress has been made in developing means of transport since their inception. Naturally, these new means encroach on the business that the railroads would like to handle, but, if the railroads cannot do the work better or cheaper than the new means, they will be passed by just like the canal-boat, the freight wagon, and the stage coach a century ago. The railroads, however, are still the backbone of this country's interchange of goods. They should be free to use public ways by land or water if they can compete with the open competition naturally inherent to such ways. For instance: If a train-load of goods stands at St. Louis, consigned to New Orleans for export or for domestic distribution; and if waterway transport by means of the magnificent channel maintained for the public by the Government is enough cheaper than rail transport to pay a railroad for installing appliances at both ends of the route to transfer cars between land rails and rails on car-floats, said floats and their propelling power being the property of the said railroad, then, the railroad should be allowed to establish such facilities, not as a monopoly, but on the same basis as railroad trucks are now allowed to distribute goods *via* city streets. If this type of transport is cheaper than the established rate by rail, it should be the base for the standard rate to be applied to goods of the class that can admit the slower rate of movement. A rate of 80% of the rail rates is unscientific and absurd.

Motor vehicles adapted for use on highways have changed the present business status. This may be called an oil-traction age. If railroad heads would give more effort to develop oil-electric locomotives to replace steam traction instead of expending their energies in the contest to be the "big boss" in the preparation for consolidation by straw corporations and other means, the case of the railroads might look better.

BAXTER L. BROWN,[23] M. AM. SOC. C. E. (by letter).—In accepting the Hoover Medal on April 8, 1930, President Hoover stated:[24]

> "With the development of our great national tools, our engines, our railways, our automobiles, our steamships, our electric power and a score of other great implements, together with the supplies of material upon which they depend, the engineer has added vastly to the problem of government, for government must see that the control of these tools and these materials are not misused to limit liberty and freedom, that they advance and do not retard equality of opportunity amongst all our citizens. * * * But when the problems which these great tools create come to the door of government, they are at once emotional problems, * * *. Our greatest difficulty in dealing with these problems of government is when the emotion comes first. Facts and the technical knowledge come but slowly or are often lost in a sea of embittered controversy."

Transportation has been one of the greatest problems of mankind since the earliest days and will so continue until the end of time. Competition of the various methods of transportation is the best incentive for its constant improvement, but such competition must be along fair and just lines, or it will be "misused to limit liberty and freedom" and "retard equality of opportunity amongst all our citizens." It was this thought that caused the passage of the Sherman Act.

Owners of the best and shortest routes, where the cost of transportation is naturally less than by any other route, should be protected in their advantage and allowed to profit by their foresight, in being the first in the field, or, by their sagacity, in securing the best available facilities. Obviously, they will get the "cream" of the business, and competing lines must be satisfied with the less desirable class of business, and they must handle it for a smaller margin of profit. It would be palpably unjust for the Government to interfere and endeavor to reduce or remove the natural advantage enjoyed by the owner of the best system, so that others could continue in operation.

---

[23] Cons. Engr., St. Louis, Mo.

[24] *Mechanical Engineering,* May, 1930, p. 528.

"Equality of opportunity" does not mean that any individual should refrain from exercising his talents for his own benefit, in order that another person less fortunate in ability should have a better chance to acquire the desirable things in life; nor, having possession of these better things, whether acquired through his own exertion or by inheritance, is it necessary for an individual to relinquish these advantages, for the benefit of others less energetic or less far-sighted than himself or than those who have gone before him.

The daily newspapers of April 26, 1930, reported that the House of Representatives had passed the River and Harbor Bill, carrying $15 000 000 for improvement of the Missouri River between Kansas City, Mo., and Sioux City, Iowa. It is questionable whether all the freight carried on the river between these two points during the period, 1930 to 1950, will be worth that much. Does any one believe that a proposition to spend that amount of money on that stretch of river for the purpose of putting it into condition to carry freight in competition with the railroads could be put before any group of financiers or bankers with the slightest hope of getting any favorable consideration?

There seems to be a disposition to bolster up unprofitable enterprises by subsidizing them by the Government, State, or city. From the standpoint of economical and efficient management, there is no reason for the existence of any institution, outside those classed as charitable, which is not self-sustaining.

Upon whether one views the relation between rail and Mississippi River transportation from the emotional or from the clearly commercial standpoint, depends the answer as to whether river transportation, as exemplified by the Government's experiment under the name of the Inland Waterways Corporation, is a success or a failure.

This governmental agency is engaging in unfair competition in the attempt to demonstrate that it is a commercially profitable enterprise. In the preparation of financial statements to be used in support of this argument, no consideration is given to certain expenditures which would necessarily be included by a privately owned enterprise, such as interest on investment, and taxes. The enterprise is profitable only to those who are so fortunately situated as to be able to take advantage of its arbitrary rate of 80% of the rail rate.

An examination of the published records of the Corporation shows that 1928 was apparently the best year, when a net income of $327 712 is indicated. If, in this year, the Corporation had been required to pay taxes in the same ratio as the railroads (6.37% of their gross revenues) its expenses would have been increased $427 272. If interest at 5% had been charged on the depreciated value of its equipment, there would have been an additional charge, to expenses, of $785,647, making a total of $1 212 919. This indicates an actual loss of nearly $900 000.

The Corporation's annual report for 1923 states that Congress had appropriated, up to 1923, nearly $750 000 000 to create navigable streams and canals.

It is not stated what portion of this could properly be allocated to the channels used by the Corporation. Interest on this sum at 5% is $37 500 000, a substantial part of which is properly chargeable to the Corporation's operation and enters into the cost of transportation furnished. It is paid by the taxpayers as a whole and not by the shippers using the service.

No part of the maintenance of the channels used is charged against the Corporation. This expense is unknown, but if it is assumed that a proper charge for this feature would be in the same ratio as the Maintenance of Way and Structures expenditures for Class I railroads for 1928 (13.71% of gross revenues), the Corporation's expense would be increased $919 608.

Attention is called to the loan of $880 000 to municipalities and the fact that the terminals of the Corporation have been in large part furnished by the municipalities served. The Corporation's Chairman, before the Appropriations Committee, mentions fourteen cities that have expended a total of $24 000 000 in providing terminals.

It is evident, therefore, that the actual costs of operating the Corporation for 1928 are under-stated to the extent of at least $1 800 000. If the traffic had been handled by rail at charges 25% in excess of those of the Corporation, the increase in cost would have been $1 677 000, or a net saving to the public of $123 000, exclusive of interest charges on funds used to provide channels.

The public, not the shipper, assumes these omitted costs, and, on a commercial basis, they must be considered in determining the cost of transportation furnished. As nothing is included for profit in this comparison, the prospect of persuading a private corporation to undertake the operation is not encouraging.

There has been a growing tendency to put the Government into business, from the operation of railroads to the introduction, in the Seventy-first Congress, of an Act to permit engineers of the Bureau of Public Roads to serve Latin-American Governments while remaining under full pay by the United States Government. This is a case of proposed subsidizing of engineers in direct competition with legitimate business and is parallel with the Government's subsidizing of waterway transportation.

The Government's experiment in river transportation has reached a point that changes the problem from one of transportation to the larger and broader one of the Government's engaging in business in direct competition with private enterprise. Is the foundation being laid for the Nation to become another Russia, with all that the name implies, or is the United States Government to continue dedicated to the protection of "life, liberty, and the pursuit of happiness?"

THEODORE BRENT,[25] ESQ. (by letter).—The general impression gained from the paper by Mr. Cornish is that the entire subject has been viewed by him with conservatism.

Concerning the demands of the country for transportation, the general rule of calculation has been that traffic demands double every twenty years, on the basis of the present growth of the United States. This only calls for a sustained increase of 3.5% annually. Despite the dislocation and inequality of the World War period, this rule has been quite well justified in the elapsed period of the decades, 1910 to 1930, and there are no present signs of a cessation of normal National growth.

---

[25] Pres., Redwood Line, Inc., New Orleans, La.

The calculations of a public requirement of 870 000 000 000 ton-miles of transportation by 1950 will probably be exceeded if the facilities are provided in time to permit traffic to expand normally. If the demand is not promptly met, then business must inevitably turn in upon itself and by a marked redistribution of industry and manufacture permit of such growth by a lessening of the volume of transportation between producer and consumer. The dislocation and capital expenditure involved in such redistribution must inevitably be greater than that necessary to enable transportation to keep pace.

The estimated cost of new railroad facilities to meet these requirements seems ultra-conservative. In its final form the paper calls for the production of about 50 000 additional miles of main track with yards, sidings, equipment, power, and permanent structures necessary to enable this and the existing railroad plant to carry the increased load. The estimated capital cost is in excess of $11 000 000 000, or slightly in excess of $220 000 per mile of new railroad, equipped. The average investment in the 240 000 miles of main-line railroad in use to-day (1930) is $125 000 per mile. The present average traffic density is 1 800 000 ton-miles per mile of road. With the addition of the 50 000 miles of new main track, the then existing plant of 290 000 miles of main line, equipped, has to support a traffic density of 3 000 000 ton-miles per mile. Obviously, this new construction will not be an average railroad. A cost of $300 000 per mile, or a total expenditure, equipped, of $15 000 000 000 will be more in keeping. The Pittsburgh and Lake Erie Railroad, which is the road of greatest density per mile in the country (10 700 000 ton-miles per mile), has cost $586 000 per mile. Making allowance for surplus equipment, the actual plant producing these results has cost about $400 000 per mile.

The new construction will not be where construction is cheap, but where it is most expensive, and freight traffic will have to bear the brunt of it. Other forms of transportation will increasingly drain away the passenger traffic and that which remains must become more and more expensive, without creating the ability to increase fares or service charges materially. An estimate of $15 000 000 000 is nearer the mark than $11 000 000 000 in the writer's survey of the situation.

The increase of 10% annually for river traffic for the twenty-five years following 1930 is conservative. The relatively small growth of traffic on the Lower Mississippi River during the operation of the Federal Barge Line in the three years, 1926 to 1929, is due solely to the delay of the Government in providing needed facilities and the relatively meager power of its incomplete joint relations to attract traffic. Both these handicaps have recently been remedied and the country is due to see a marked upward trend of ton-miles at least for the 10-year period (1930 to 1940).

It is possible to measure the maximum capacity of a waterway which has limiting physical features, such as locks; but who can measure the capacity of a well-regulated river, such as the Lower Mississippi? Three tows per week in each direction between New Orleans, La., and St. Louis, Mo., produced about 1 338 000 000 ton-miles of transportation in 1928. Loaded to capacity both ways, these tows could as readily have produced 3 000 000 000 ton-miles. Tows of this size could be dispatched at half-hour intervals and be less noted than the daily parade of lake freighters in front of Detroit, Mich. Such service would produce 300 000 000 000 ton-miles of transportation twice the estimate given by Mr. Cornish and still the capacity of the river would in no manner be taxed. The estimate of the carrying capacity of this river, at least, is conservative.

The cost figures on most inland waterways are still experimental. Projecting present known costs on the Lower Mississippi to a basis of 2 500 000 000 ton-miles, the cost of operation, including depreciation of plant and terminals, is 3.27 mills per ton-mile. Profitable operation in the carriage of bulk commodities at 1 mill per ton-mile on the Lower Mississippi River will be reached some years before the inland waterways are called upon to carry burdens comparable with the 60 000 000 000 ton-miles which this study forecasts.

The day of cheap railroad transportation is past. Sweeping increases in rates are being made in all important territories to bring railroad revenues to the mark fixed upon as necessary to support and expand the plant.

Shippers are only beginning to realize at what low costs good transportation by water can be performed at a profit. The cleavage between the relative cost and the value of the service increases year by year. The very force of necessity is bringing the entire rail-water transportation problem rapidly from the academic to the experimental stage. There will always be a field for controversy, but with growing appreciation of an informed public the assertions which to-day require "damnable iteration" will take the form of truisms. For the time being, however, it is necessary to go on proving, over and over again, that water transportation is cheap and rail transportation is very expensive. Mr. Cornish does this in a masterly way. This research proves the old truth in a new way and is a valuable reference work.

SAMUEL O. DUNN,[26] ESQ. (by letter).—The National Administration has proposed, as one of its major activities, the widespread development of inland water transportation. In view of the importance to the United States of adequate, dependable, and efficient transportation service furnished at the lowest possible cost, of the magnitude of the proposed waterway development, and of the vast expenditure of public funds which such development would entail, some analysis of the general waterway situation, based upon the facts as disclosed by past experience, may very well be made.

Inland Waterway Development.—The Great Lakes are inland waterways, but it should be understood that in the following discussion the words "inland waterways", are used only in reference to improved rivers and canals. The Great Lakes were made deep and large by Nature, and nobody questions that transportation on them is much cheaper than by rail.

---

[26] Editor *The Railway Age*, Chicago, Ill.

At Louisville, Ky., on October 23, 1929, President Hoover, in connection with the celebration of the completion of a 9-ft. channel in the Ohio River, outlined the general policy of his Administration to continue "the modernizing of every part of our waterways which will show economic justification in aid of our farmers and industries." No elaboration was made of the phrase "economic justification."

There are many waterway advocates who subscribe wholeheartedly to this program and who support it with optimistic opinion, theory, belief, and prophecy. Yet in fairness to the waterways, which in the past have been embarrassed by the too optimistic predictions of their friends, in fairness to the railways with which these waterways will compete, but most of all in fairness to the general public which will "foot the bill," the basis of actual fact is the criterion by which the program should be judged.

Chief among the claims advanced in support of waterway development, and hence presumably closely linked to "economic justification", is the statement that such development will "cheapen" transportation. What is meant by "cheaper" transportation? Do the advocates of waterway development mean that waterway conveyance charges alone will be lower than the corresponding rail rates, or that total waterway costs, including both charges for carriage and the burden of capital charges, maintenance, and operating expenses borne by the taxpayers, will be less than the railway rates? These questions thus far remain officially unanswered.

In an Interim Report issued in 1927 by the Board of Engineers for Rivers and Harbors and the Shipping Board, "to render immediately available data showing the importance to the country of the improvements completed and in progress in the Ohio Valley," the following statement was made:

> "The primary measure of the economic value of any waterway system is the savings in transportation costs which it affords. Usually this saving is shown by comparing the water transportation costs with the rates for moving the same goods between the same points by rail. Included in this cost of water transportation should be the charges for interest on capital investment in waterway improvements plus annual cost of operation and maintenance."

An exactly opposite position is taken by Maj.-Gen. T. Q. Ashburn, U. S. A., Chairman and Executive of the Inland Waterways Corporation, who states, in the 1929 Annual Report of that Corporation, that,

> "Because hundreds of millions of dollars have been spent in making streams navigable is absolutely no reason why the upkeep of these rivers or the cost of their maintenance should be charged to this corporation or any other corporation operating fleets on these water 'rights-of-way'."

Finally, a middle ground also appears to be taken in the Interim Report mentioned, which claims—although with little convincing evidence—that certain waterway expenditures have already paid for themselves. In a report entitled "Inland Water Transportation in the United States", the Transportation Division of the U. S. Department of Commerce advances a theory, subscribed to by many, that much of the past expenditures for river improvements should simply be charged off and forgotten. Furthermore, it states that "for services on channels already made navigable or on canals already built the returns need cover only current operating expenses and necessary maintenance of channel, terminals, and floating equipment in order to justify continuance."

So far, search for an interpretation of "cheaper" transportation and its relationship to "economic justification" has brought to light only conflicting Government opinions. In order to obtain official information, inquiry as to the meaning of this phrase was made of the Hon. Patrick J. Hurley, Secretary of War, under whose authority and jurisdiction National waterway improvements are carried out. Mr. Hurley's only reply was that "The War Department has no intention of inviting controversial discussion as to the relative merits of various forms of transportation."

In the face of this official attitude, one must draw his own conclusions. It would appear that the only logical interpretation to be put upon the claim of "cheaper" water transportation is that it means that total waterway costs, regardless of who pays them, will be less than the corresponding rail charges. Total cost is the crucial test.

The only cost paid for the transportation of freight by rail is the freight rate charged. The cost of transportation by inland waterway is divisible into two parts. The first is paid by the public, and consists of interest upon the investment made in improving the waterway and building terminals, the expenses of maintenance, and certain operating costs, particularly in the case of canals or canalized streams. This public payment represents a subsidy to the waterways that results in an increase in taxes. The second part of the cost of water transportation is that incurred in the actual movement of freight.

Now, on sound principles of economics, what should be the objective? Should it be to reduce freight rates entirely regardless of the effect upon taxes? Plainly, if the development of waterways will result in reductions of freight rates, it will also result in increases in taxes. Total taxes in the United States increased 30% between 1923 and 1928, and amounted to twice as much as the total freight earnings of the railways. Can it be contended that taxes are so unimportant, and freight rates so important, that the National prosperity would be promoted by securing a saving in freight rates at the cost of an increase in taxes that would exceed the saving in freight rates?

Furthermore, the development of waterways is not the only way available in which taxes can be used to reduce freight rates. Recently, the British Government actually has secured reductions in freight rates on agricultural products by giving the railways equivalent direct subsidies from the Public Treasury. The railways of Canada receive subsidies from the Government for making low rates. This is not pointed out in advocacy of similar subsidies for the railways of the United States, but it is surely pertinent to call attention to the fact that if the total cost of inland water transportation, including the part paid in taxes, is or will be greater in this country than the total cost of providing similar transportation by rail, then any desired reduction of freight rates, either on certain commodities, or in certain parts of the country, or even on all traffic throughout the United States, obviously could be secured at a lower cost to the taxpayers by having the Government subsidize the railroads than by having it develop and maintain waterways. As regards the total cost of carrying all freight there is this further fact to consider: The greater the density of railway traffic, the cheaper the railways can handle each unit of this traffic. Therefore, any policy that hinders the increase of railway traffic also hinders the reduction of the cost of transportation by rail.

The situation thus narrows down, primarily, to a question of relative costs. How, then, do the total costs of transportation by inland waterway and by railway in this country compare, and how may they be expected to compare in the future? Actual experience can never be intelligently or safely disregarded. For many years the Government has been improving inland waterways and the country has actually had transportation by both water and rail. There is an abundance of information regarding the costs to the public of rail transportation, but lamentably little reliable and satisfactory information regarding costs of inland water transportation. For many years the Army Board of Engineers for Rivers and Harbors and others have been giving estimates to Congress and the public regarding proposed waterway improvements, including the traffic that would be handled, the expense of improving and maintaining the waterways, and the savings in transportation costs that would result. Those who have made these estimates seldom have come forward after the improvements have been made with figures showing the extent to which results have or have not justified their estimates and expenditures.

There are three great inland waterways in the United States upon which transportation operations are now being conducted. These are the New York State Barge Canal, the Ohio River System, and the Mississippi. What does the record show?

In 1929, the New York State Barge Canal handled a freight traffic amounting to 715 551 472 ton-miles. Cost of canal maintenance and repairs amounted to $3 390 432, while interest on the investment, at 5%, amounted to $8 811 061 in that same year. On a ton-mile basis, maintenance and repair costs were 0.474 cent, and capital costs, 1.231 cents, a total "hidden" cost of 1.705 cents per ton-mile which was paid in taxes by the people of New York State. The average actual waterway conveyance charge on the Barge Canal—a figure which far too often is considered by the waterway advocate as the total cost of water transportation—is conservatively estimated at 0.450 cent per ton-mile, making the total cost of transportation on the New York State Barge Canal 2.155 cents per ton-mile. The corresponding average ton-mile revenue of the Class I railways in the Eastern District in 1929 was 1.090 cents. Frederick Stuart Greene, M. Am. Soc. C. E., Superintendent of Public Works of New York State, has said that in 1925 it cost the State $4.51 per ton for all freight loaded on the canal, regardless of length of haul, and that it would have been cheaper for the State if all the freight carried on the canal had been put in railroad cars and the State had paid the freight bills. Is this "cheaper" transportation, or economic justification?

What of the Ohio River System? It is conceded even by some of the principal advocates of waterway development that the bulk of the very large expenditures heretofore made on waterways has been wasted, because it has not resulted in the increase of water traffic and the reduction of transportation costs which it was intended to accomplish. It is stated, however, that future expenditures will be made more wisely and that it is entirely beyond question that they will result in cheapening transportation. In other words, the public is asked to do what is always unintelligent and unsafe, namely, to disregard experience. Surely, however, it cannot reasonably be asked to disregard very recent experience. The completion of the improvement of the Ohio River System is so recent that it was celebrated within the past year (1929). What does this waterway show on a cost basis?

Some very interesting figures of "savings" to the public resulting from water-transportation operations on the Ohio River System are contained in a report entitled "Transportation in the Mississippi and Ohio Valleys", issued in 1929, by the War Department Board of Engineers for Rivers and Harbors and the Bureau of Operations of the U. S. Shipping Board. In this report the following statement is made: "Based upon a ton-mileage of 2 649 000 000 on the Ohio River System in 1926, the savings in transportation costs amounted to about $25 000 000."

From the foregoing figures, the "savings" claimed appear to amount to 0.944 cent per ton-mile. Disregarding the fact that the ton-mile basis of comparison is peculiarly favorable to the waterways because of the fact that water routes, over any material distance, are almost invariably much more circuitous than the corresponding rail routes, it is interesting to note that in the same year (1926), the average ton-mile revenue of the eight principal Class I railroads operating in the Ohio Valley was only 0.883 cent. In other words, the waterway "savings" claimed amounted to 107% of the corresponding average rail revenue per ton-mile. At their average ton-mile rate of 0.883 cent, the adjacent railways would have carried the entire traffic moved on the Ohio River System for less than $23 400 000, or for about $1 600 000 less than the saving in transportation costs claimed.

This average of 0.883 cent per ton-mile for the adjacent railways covers all classes of traffic and is thus increased by the inclusion of much high-grade traffic which the waterway does not handle. The greater circuity of river routes also distorts the figures in favor of the waterway. On the other hand, average hauls on the Ohio River System are relatively short and, on corresponding rail hauls, average ton-mile revenue would be affected by the relatively large part of terminal expense it would have to bear. These three factors cannot be evaluated statistically. Despite this fact, it seems pardonable to view with some skepticism statistics which indicate that the transportation of freight on the Ohio River System resulted in savings to those for whom the freight was carried actually exceeding the total amount that the railways, at their average revenue per ton-mile, would have received for handling it.

From the report of the Board of Engineers previously mentioned, the total investment of the public's money in the Ohio River System appears to be about $240 000 000. A study of all the available data indicates that for every ton of freight carried 1 mile on the Ohio River System the taxpayers are paying a cost of about 5 mills; and those whose freight is carried are paying a cost of about 5.5 mills, or a total of more than 1 cent, as compared with an average ton-mile revenue of less than 9 mills on the adjacent railways. Approximately 90% of the traffic handled on the Ohio River System consists of such low-grade commodities as coal, sand, and gravel. The average revenue per ton-mile of the Chesapeake and Ohio and the Norfolk and Western Railroads, which operate in the same territory and the traffic of which consists mainly of low-grade commodities, especially coal, is only about 6.5 mills, or 40% less than the average total cost of transportation on the Ohio River System. Does this show "cheaper" water transportation, or economic justification for further waterway development?

Finally, what is the situation on the Mississippi River, the third of the Nation's great waterways? It is impossible to obtain detailed statistics regarding the Government's investments in the Mississippi for navigation purposes, or detailed figures regarding operating costs, but the situation seems very aptly summed up in the report of the Inland Waterways Corporation for 1929, wherein General Ashburn states:

> "If the cost of their [the rivers'] upkeep be charged to the few operators thereon, the only result would be to destroy their value as water 'rights-of-way' by raising the cost of transportation *via* water so high that no carrier could operate and the whole policy of Congress would be defeated."

Clearly, if the imposition upon carriers by water of charges merely to defray the cost of maintaining the waterways would make the cost of water transportation so high that the water carriers could not compete with the railways, it must be that the total cost of transportation by inland waterways is, and will be, greater than the cost of transportation by rail. In other words, water transportation is made possible only through large Government subsidies, paid, in turn, by the people at large in the form of taxes.

So much for cost! What about certain other requisites essential in an efficient transportation machine? To be specific, what about adequate and reliable service?

The following quotations from the 1929 report of the Inland Waterways Corporation are highly illuminative as to the ability of waterways to furnish adequate and dependable service.

Regarding the Upper Mississippi, the report states:

> "The rainfall during the months of May and June was very deficient and by mid-June we were beginning to have trouble on account of low water. * * * Our greatest misfortune during the month of August occurred when the lower gates of the lock at the high dam between the cities of St. Paul and Minneapolis gave way, permitting the water above the dam to escape and leaving the steamer *Altair* and six barges hopelessly stranded at the Minneapolis terminal."

Similar misfortunes visited the Lower Mississippi. Witness the report:

> "Within the last five months of 1929 we encountered every detrimental navigating element presented in the almost eleven earlier years of operation and in an aggravated degree, resulting in serious interruptions to schedules and revival of the service complaints which had been almost entirely eliminated. The inadequate channel was comparable to 1922 with the exception that in 1929 a far greater area was involved and the duration of the low-water period was almost twice as great. In 1929 the detention to line vessels in active service directly chargeable to fog and unavoidable channel obstructions was as follows: August, 1 913 hours; September, 3 356 hours; October, 3 323 hours; November, 743 hours; and December, 2 047 hours. Ice closed the St. Louis harbor almost an entire month earlier than usual."

As to the third operating division of the Inland Waterways Corporation, the report continues:

"On the Warrior River we experienced the greatest flood period in our career, considerably slowing up operations. Normal tows of seven barges had to be reduced to two and three barges, in order that the power might carry them up stream."

As regards the Ohio River, the following Associated Press dispatch was sent from Portsmouth, Ohio, on June 5, 1930:

"Boat traffic on the Ohio River between Pittsburgh and Cincinnati was abandoned temporarily to-day when the water dropped so low as to make navigation impossible. Lack of rainfall during the past several weeks was blamed. The *Chris Greene* went aground on a sand-bar above here last night, but succeeded in breaking loose and later attempted to return down stream to Cincinnati. A number of boats and packets tied up here rather than attempt passage through the sand-bars. The navigable stage here is 9 ft., but the level had dropped to 6.5 ft. to-day and was dropping rapidly. Above Portsmouth the water became so shallow that persons could wade across the river. The water dropped below the navigable stage at Dam 30, Greenup, Ky., and Dam 32, Vanceburg, Ky. Dam 31, near here, was of no use to river traffic as wickets had been raised several days ago to make repairs which will require three weeks. The *Senator Cordell,* bound from Cincinnati to Pittsburgh, tied up here because of the low water. Boats that stopped here were the *Ed Kenna,* pulling empty barges from Cincinnati to Huntington; the *Otto Market* and the *A. C. Ingersoll,* both bound to Cincinnati with tows of coal."

No further evidence is needed, beyond these official statements, to indicate the weakness inherent in waterways when it comes to providing adequate and dependable transportation. Whether from ice, from low water, or from high water, streams are affected by natural forces beyond the control of the engineer, be he ever so skilled. Reliable and adequate transportation service is essential to National life, in its present highly developed industrial stage. Such service the railways give to every section of the country. Such service the waterways do not, and cannot, give.

The increase which has been made in the past decade in the efficiency of railway operation—marked by cheaper service, faster service, more adequate service, and more dependable service—has been characterized as one of the outstanding achievements of the post-war era. Business leaders of the Nation have testified at length as to the value of this improved rail service, both to their own industries and to the Nation at large. This testimony is a matter of common knowledge, including in its list of benefits such factors as greatly reduced inventories; release of capital frozen in transit; reduction of factory operating expense, and stabilization of labor forces; reduction of demurrage charges; receipt of shipments in better condition; increased volume of sales; and the widening of potential markets. In fact, it has been reliably estimated that the benefits derived by general business from the improvements made in railway service since 1922 are equivalent in annual value to a reduction of at least 30% in present railway freight rates. Do the waterways offer any such service benefits to the Nation? They do not and, from their natural characteristics, they can not.

In determining "economic justification" the fact cannot be disregarded that rail service is faster, more dependable, and better in other respects than is the transportation service offered on inland waterways. Therefore, to put the water service on an economic parity with rail service, the total cost of transportation by water must be enough less than the cost by rail to offset this difference in the character and value of the service rendered. It has often been stated—and, indeed, used as a basis for calculating water rates—that rail service is at least 25% more valuable than water service and that, therefore, the costs by water must be at least 20% less than by rail to offset this difference in the quality of service.

The available facts regarding the three principal inland waterways indicate that the total costs of transportation thereon are actually greater than the corresponding rail charges. They are thus in the position of furnishing poorer service at higher costs. Is this "economic justification" for further waterway development?

Finally, there is one further point upon which question may be raised as to the advisability of further waterway development. In the first place, these proposed improved waterways, if developed, either will or will not handle a material tonnage. If they will not, there is no legitimate reason for spending vast sums of public money on their improvement. If they will handle a material tonnage, this, of necessity, must be taken in large part from the railways. That the waterways, as their advocates claim, will develop any considerable volume of "new" tonnage is hardly susceptible of proof. This is not to deny that the development of new waterways, under some conditions, may result in increased traffic for certain railways adjacent thereto; but in the main it appears that such problematical increases in traffic on certain railways may result only through the relocation of industries, to take advantage of the possible lower water conveyance charge and to profit from the public transportation subsidy financed by the taxpayers. Under such circumstances, traffic gained by one group of railways will be traffic lost by another rail group. It is impossible to avoid the conclusion that if improved inland waterways attract the traffic which their advocates foresee, there will be a material reduction in the tonnage that would otherwise be offered to the railways for transportation.

What will this mean to rail costs and rail service? The inability of inland waterways to furnish adequate and dependable service has already been mentioned. This inability means that the railroads, as an economic necessity, must maintain surplus transportation capacity to be able to handle adequately, dependably, and efficiently the waterway traffic (or a considerable part of it) at those periods when the waterways, from causes beyond human control, cannot furnish adequate and dependable service. Such periods have occurred times without number in the past. That they will recur is beyond question.

Meanwhile, what of the railways in those years in which the waterways are able to furnish their service and in which traffic is attracted from the rail lines because of the lower direct conveyance charge to the shipper? What of the capital costs on the railway plant? How will their properties, necessary in time of waterway failure, be supported under a reduced tonnage? It is obvious that this remaining rail tonnage must either bear higher rail charges or else suffer impaired rail service. In either case the same practical effect is produced—the cost of rail transportation is increased.

This economic necessity for railroad transportation capacity to assume the carriage of water traffic in those inevitable periods when the waterways cannot function satisfactorily is in striking contrast to the second argument advanced by the waterways advocate—that further waterway development is required to "relieve" the railways, either of current or of probable future traffic demands upon them. This argument, at least in part, was made by Mr. Cornish. Without attempting any detailed analysis of his paper, some observations upon his conclusions, as viewed in the light of actual traffic changes on the railways in the forty years since 1890, may pertinently apply not only to his arguments, but to those of others who foresee a railway need for "relief" from an overwhelming traffic.

His conclusions are predicated upon a forecast that in 1950 railway freight traffic will reach the staggering total of 870 000 000 000 ton-miles. Revenue ton-mileage in 1929, the highest figure ever reached, was approximately 450 000 000 000; so Mr. Cornish apparently foresees an increase of 420 000 000 000 ton-miles in the next 21 years. What in the record of the past justifies such a belief? What are the facts?

Statistics of railway revenue ton-mileage are available for the past forty years. From 1890 to 1929 (39 years) the increase in railway freight traffic was 374 000 000 000 ton-miles. Mr. Cornish's estimate for the next 21 years is, therefore, 12% in excess of the actual increase in the entire last 39 years. The increase in the 21 years from 1908 to 1929 was 232 000 000 000 ton-miles. Mr. Cornish's anticipated increase is, therefore, approximately 80% in excess of the increase that actually occurred within the last 21 years.

Even more significant is the marked change which has occurred since 1920 in the trend of railway freight traffic, a change characterized not by an increasing growth, but by a rapidly declining rate of growth. From 1890 to 1900 there was an annual average increase in railway ton-mileage amounting to 6 500 000 000. In subsequent periods the average annual increase was as follows: 1900-1910,. 11 500 000 000; 1910-1920, almost 16 000 000 000; 1920-1929, only 4 000 000 000. Thus, during the last nine years, the average annual increase in railway ton-mileage was only about one-fourth as great as in 1910-1920, only about one-third as great as in 1900-1910, and only about two-thirds as great as in 1890-1900. In the light of this marked retardation since 1920 in the rate of growth of railway freight traffic, it seems extremely doubtful whether the increase within the next 21 years will exceed 125 000 000 000 ton-miles, which would make the total freight traffic available for the railways in 1950 only 575 000 000 000 ton-miles. Mr. Cornish estimates that "railroad renewals and increased operating efficiency may provide" for the handling of about 650 000 000 000 ton-miles in 1950. It would appear, therefore, from the actual experience of the past decade, and Mr. Cornish's own estimate of the amount of traffic that can be handled by rail in 1950, that the railways not only will not need any "relief" by waterways in 1950, but will be able then, as they are now, to handle much more traffic than is available for them.

The railways have no fear of an overwhelming increase in traffic, in view of the experience of the last decade. On the contrary they fear, and with reason, that the retardation in the rate of increase in their freight traffic, caused at least in part by the loss of business to their subsidized competitors, may result in a situation wherein it may be difficult to support their properties. Already retrenchments in capacity have been made in various parts of the railway plant. On the Class I lines the number of passenger-train cars has been reduced from 55 040 in 1924 to 53 097 in 1928; the number of locomotives, from 65 358 in 1924 to 59 391 in 1928; total locomotive tractive power, from 2 611 000 000 lb. in 1926 to 2 580 000 000 lb. in 1928; the number of freight-carrying cars, from 2 357 234 in 1925 to 2 297 677 in 1928; and the total carrying capacity of freight cars, from 105 953 000 tons in 1926 to 105 327 000 tons in 1928. Further, in the peak year of 1929, the average daily freight-car surplus at no time fell below 107 000. In the face of these figures it seems obvious that the railways' imperative need is not for "relief" from traffic, but for more traffic. Every ton taken from the railways increases the unit cost of rail transportation; every ton gained by them reduces this unit cost.

It thus appears from the available evidence that on the three great inland waterways the cost of transportation is greater than by rail; it appears that inland waterways are physically incapable of furnishing adequate and dependable transportation service; it appears that if the proposed development of inland waterways attracts to those waterways the volume of traffic which is predicted, there will be a material loss in the volume of tonnage which otherwise would move by rail, while at the same time the railways must maintain surplus transportation capacity against the inevitable waterway breakdowns; it appears that there is no necessity for waterway development to "relieve" the railways. In consequence, waterway development will mean that the traffic which these waterways may attract will be handled at a higher cost to the Nation in general than if it were carried by rail; that this service will be inferior to rail service; and that either railway rates will have to be increased on the remaining rail traffic or the quality of rail service reduced. In other words, the only results of further inland waterway development seem to be increased National transportation costs and impaired National transportation service.

Those who favor the development of waterways doubtless will answer that the facts presented herein do not prove the greater cost of water transportation as compared with rail transportation. If so, let them come forward with better data—data such as would be convincing to any trained student of business and economic problems. The most outstanding fact regarding this controversy concerning the comparative costs of water and rail transportation is the lack of credible and convincing evidence in support of the claims made as to the low and prospective costs of transportation on inland waterways. The public is being asked to invest hundreds of millions of dollars in waterway improvements, and in behalf of this expenditure claims as to the cheapening of transportation that will result are being made, which are not supported either by the facts derived from past experience or adequate data as to future probabilities.

The Nation has to-day a railway system which is being operated efficiently and is more than adequate to all the demands being made upon it. When it is proposed to develop another system of transportation by the expenditure of public money, which, it is claimed, will carry freight at less cost than the existing means of transportation, the burden of proving that claim obviously rests upon those who make it. As this means of transportation is to be provided by increasing the taxes of the public, it is obvious that its development cannot be economically justified except by showing that it will probably, and, indeed, almost certainly, cause a reduction in what the public must pay for freight transportation that will exceed the increases in taxes it must pay.

To say that the development of waterways will "cheapen" transportation because it will cause part of the Nation's freight to be carried for lower rates than it would be carried by rail simply evades the real issue. The cost of transportation, whether by airway, highway, waterway, or railway, is the total amount that has to be paid to provide such transportation, regardless of who pays all or any part of this cost. All the people directly or indirectly pay both taxes and freight charges. The question of the expediency of developing any inland waterway always, therefore, is a question of taxation as well as of freight charges. It always involves the questions, first, as to whether the increase in taxes will or will not exceed the savings in freight charges directly effected by the development of the waterway; second, as to whether there will not result a diversion of traffic from the railways that will make it necessary for them to charge higher rates on their remaining traffic than would otherwise be necessary; and, third, as to whether, if there actually is a net reduction in the total amount that must be paid for transportation by both rail and water, those who pay the increased taxes will be the beneficiaries of the reduction in freight rates.

Not until the advocates of inland waterway development squarely meet these questions with data such as would be convincing to private capitalists who were being solicited to invest their own money, will inland waterway development be discussed on its merits as a National economic and business problem. The American public is being urged to invest a large amount of its capital in inland waterways to increase the prosperity of the Nation, and not merely that of certain shippers, communities, or territories. It ought not to invest it, or be asked to invest it, on the basis of information any less complete and convincing as to probable costs and savings than that which would be demanded by business men who were being asked to invest their own capital.

This country has had more than a hundred years of experience with inland water transportation. Would a private business, after a hundred years of experience in its own field, spend a dollar for development on prophecy? Would it not search the records of the past for a guide to the future? Is it unreasonable to ask similar prudence in Government expenditures?

"Economic justification" is an adequate reason for future waterway development in the United States; but in accepting this reason it would appear, first, that the term should be interpreted as a reduction in total National transportation costs; and, second, that it be supported not by belief, opinion, theory, or prophecy, but by facts and facts alone.

HENRY W. HOBBS,[27] M. AM. SOC. C. E. (by letter).—The discussion of this subject has raised the question of the economic justification of inland waterway improvement at Federal expense. Before the construction of railways, rivers and canals provided the best, and in many cases the only, means of transportation. They continued to be the primary means of transportation up to about 1860, the early railroads acting mainly as feeders to the waterways. With the extension of the railroads severe competition developed between them and the waterways, as well as among the different rail lines. The effect of this competition is still seen in the rail rate structure of the country, with its marked discrimination in favor of points with water competition. The improvement of these earlier waterways was fully justified, as they formed the primary means of transportation and were important factors in the development of the United States.

In the latter half of the Nineteenth Century there was a period when many waterway improvements were authorized, some of which were of questionable value. To-day, waterway legislation is probably subjected to more careful study than any other question coming before Congress. In 1902, Congress created the Board of Engineers for Rivers and Harbors, to which all projected waterway improvements are referred for study and recommendation. Each project is now submitted for study to an officer of the Corps of Engineers who is stationed in the general locality, where a district office is maintained. He first submits a preliminary examination report, which contains such economic data as he is able to collect, with a general description of the locality and the details of the improvement desired. This report is then acted upon by another officer of the Corps of Engineers, who, as Division Engineer, has charge of a number of districts. The District Engineer's report, with the comments of the Division Engineer, is next studied by the Board of Engineers for Rivers and Harbors. If it finds that the proposed improvement appears economically justified, a field survey is made to determine the cost of the work and such other features as appear necessary in reaching a final conclusion. The survey report is made by the District Engineer, receives the comment of the Division Engineer, and the case is again considered by the Board of Engineers for Rivers and Harbors.

In the office of the Board, engineers, statisticians, economists, and rate clerks assist in working up the information received from the District Engineer and from other sources. This includes data concerning the potential commerce and the points of origin and destination of the several commodities. Much information of this character is obtained from the Department of Commerce. The practicability of the expected movements is studied, together with the available means of shipment and the transportation costs. The prospective savings in transportation costs are the differences between the existing rates on the same commodities, between the same or equivalent points, and the cost by an improved channel. The transportation costs by water include all terminal and transfer charges. An unfavorable report is indicated if these savings are not sufficient to show a considerable margin above the annual carrying charges on the proposed improvement. These charges include interest and amortization on the investment in channel improvement, and the maintenance cost, including the operating cost of locks and dams if such works form part of the project. No account is taken of savings to the public resulting from any possible rail-rate reduction to water competitive points. The principal traffic of the waterways is made up of bulk commodities, and little consideration is given in the Board's studies to the movement of high-grade general freight. About two-thirds of the recommendations made by the Board to Congress are unfavorable.

---

[27] Executive Secy., Board of Engrs. for Rivers and Harbors, Washington, D. C.

The claim that water transportation is more expensive than rail appears to be based on a comparison of water-haulage costs, including carrying charges on the improvement, with the average revenue per ton-mile of the railroads serving the territory. Taking into consideration the difference in the character of the shipments by water and by rail, and particularly taking account of the many peculiarities of the rail rate structure, it is apparent that such a comparison does not give a correct result. It appears that the only fair way to arrive at the saving in moving a particular commodity by water rather than by other means is by a direct comparison of the rates on that particular commodity between the same or equivalent points. To illustrate this, the average cost of moving coal on the Monongahela River is about 3 mills per ton-mile, while the rail rate on coal on parallel roads, for the same average haul, is 19 mills per ton-mile. This ton-mile cost on the Monongahela River cannot be used on another stream without making corrections for channel dimensions, current velocity, distances between the origin and destination of the commodity, size of cargoes, regularity of shipments, back haul, and other conditions peculiar to the river under consideration. Wheat is moved by the Inland Waterways Corporation from St. Louis, Mo., to New Orleans, La., at a rate of 2 mills per ton-mile, or $1.30 per ton under the rail rate.

The cost to the people of the United States of making waterway transportation possible is covered by taxation. While the total taxes in the United States have increased greatly since 1923, Federal rates of taxation have been several times reduced during the same period. The returns which the people receive from this taxation for waterways come in the form of reduced commodity prices. The savings to industries, due to the lower cost of transporting raw materials by water, make possible the marketing at lower prices of the products of the industries affected. The iron and steel industry may be noted as one of those affected. These commodities enter into the construction of many things in general use, such as household utensils and equipment, thereby distributing the benefits to the entire population.

The cost of moving coal on the Monongahela River has been cited. The annual carrying charges on the improvement of the Monongahela River, based on the ton-mileage of 1929, are less than 1¼ mills per ton-mile. The carrying charges on the improvement of the Ohio River are greater than this at the present time, but will decrease as the traffic increases. Taking the Ohio River System as a whole, the total ton-mile movement in 1929 was 2 886 436 000. The savings on the Monongahela River alone are in excess of the annual charges for the entire Ohio River System. The Ohio River improvement, although only recently completed, already shows returns slightly in excess of the annual costs.

The traffic on the system in 1929 totaled 59 537 000 tons, an increase of about 10 000 000 tons over that for 1925. The activities of large concerns, which have expended many millions of dollars in building towboats and barges, indicate that the movement of commerce on the Ohio River System will continue to increase. The total commerce on rivers and canals in 1929, exclusive of the Great Lakes and not including rafted timber, was 170 688 000 tons, an increase of about 35 000 000 tons over 1925. The ton-mileage was 9 446 712 000; in 1925, it was 8 417 052 000.

After a waterway has been completed and has been in operation a sufficient length of time to indicate its value as a part of the transportation system of the country, careful studies are made to determine the justification for the expenditures made by the Federal Government. In a few cases negative results have been shown. The Cape Fear River above Wilmington, N. C., has not shown a profit, as the improvement does not extend far enough up the river to give adequate depth at Fayetteville, the center of the principal tributary area. The St. Johns River, in Florida, between Jacksonville and Palatka, shows a small deficit, although the section of the same river between Palatka and Lake Harney shows a large profit. The only stream on which a considerable deficit has been shown is the Warrior River, in Alabama. Water transportation on this stream was handicapped for a number of years by an unfair division of rates between the rail and water carriers. Considered as a whole, the waterways of the country return each year, in reduced transportation costs, a sum much greater than the carrying charges on the improvements.

Dependability of service is an important matter. The length of the navigable season on the Great Lakes is about eight months. During the closed season it has not been found necessary to continue shipments of the principal commodities by rail. Large quantities of these bulk commodities are stored during the navigation season to preclude the necessity for such movements at higher cost during the closed season. The Ohio River is now open to navigation throughout the year, being subject in general to unimportant delays averaging not to exceed ten days annually. Such delays are not of serious importance on transportation lanes carrying principally bulk commodities, and instances can be cited where delivery is made more promptly by water than by rail.

Some classes of freight move most cheaply by water, others by rail, and others by motor truck. The greatest economy of transportation can be obtained by a proper co-ordination of these various means of transportation. It would be an economic loss to do away with the cheaper forms of transportation, whether they be by waterway, by railroad, or by truck, in favor of movement at higher cost over any other of these facilities. An ideal condition would provide for shipments, by the various routes, of the particular commodities best suited for transportation over each.

C. W. KUTZ,[28] M. AM. SOC. C. E. (by letter).—In 1925, the writer prepared a paper on the "Relation of the Ohio River and Its Tributaries to Transportation in the United States."[29] At that time, the 9-ft. project was incomplete, and the tonnage moving on the Ohio River itself, exclusive of tributaries, was 15 725 849. To determine the water-haulage cost of moving this tonnage and its relation to published rail rates on like commodities that have the same points of origin and destination, a comprehensive analysis was made of the 1925 tonnage.[30]

---

[28] Brig.-Gen., U. S. A. (*Retired*), Washington, D. C.

[29] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1105.

Nineteen separate movements were studied and for each is shown the rail rate and the water-haulage cost as secured from the principal users of the river. Some of these users were common carriers and some contract carriers, but the large majority were private carriers. In the first two classes the water-haulage cost included an item of profit to the owner. In the third class the water-haulage cost included a 6% return on the investment in floating plant and on the privately owned terminal facilities, a return which is comparable to that which the railroads of the United States are permitted, under the Transportation Act, to earn on their investments,

As most water movements involve higher terminal charges than corresponding rail movements, the difference between the rail rates and the water-haulage costs was reduced by the estimated terminal differential. This method of determining economic justification is superior to a comparison of cost per ton-mile, as it eliminates differences due to the generally greater length of the water route, and it also avoids the faulty comparison of a long-haul rail rate with a short-haul water rate. In 1925, the average length of haul on railways was in excess of 200 miles, whereas that on the Ohio River was less than 90 miles.

It may be noted that this table (Table 23[30]) dealt only with the Ohio River, excluding all its tributaries, some of which, such as the Monongahela River, are enormously profitable, while others are economic failures. When the movement involved the use of a tributary, as well as the Ohio River, the difference between the rail rate and the water-haulage cost was prorated between the two streams on the basis of the mileage of each involved in the movement. The table showed that the differences between rail rates and water-haulage costs, when applied to the 1925 tonnage, resulted in a saving to the shippers for that year of $5 328 594, which was about 87% of the annual charges for 1925.

TABLE 9.—COMPARISONS BETWEEN TONNAGE, TON-MILEAGE, AND SAVINGS

Year	Tonnage	Ton-Mileage	Savings
1925	15 725 849	849 195 846	$5 328 594
1926	19 754 978	1 101 629 085	6 911 600
1927	20 128 518	1 214 490 565	7 098 600
1928	20 938 267	1 344 687 000	7 796 000
1929	21 955 000	1 512 584 000	8 174 106*

* Estimated, increase being proportioned to the increase in tonnage.

The project was completed in 1929 at a capital cost of about $115 000 000. The annual cost of operation and maintenance was then estimated at $2 350 000.

---

[30] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), Table 23, p. 1178.

If interest at 4% and a depreciation or obsolescence factor based on an assumed life of 50 years (sinking-fund basis, 4%), is added, the annual charges of the completed project will be equal to $7 703 000.

Table 9 shows how the commerce and the savings, based on similar analyses, have increased since 1925. From this table and from the estimated annual charges, it appears that the canalized Ohio River, although just completed, is already yielding savings to shippers in excess of the annual charges. As its capacity is many times greater than the present tonnage, the economic justification of the investment seems to be assured.

T. Q. ASHBURN, ESQ. (by letter).—Too much has been said in the various discussions of this paper about the relative merits of rail, water, and motor transportation and the effect of such competition upon the revenues of the various carriers, and far too little about the interest of the public in securing the best transportation at the cheapest possible rate. Every citizen is vitally interested in the cost of transportation because it is concealed in the price of every article he utilizes, the food he eats, and the clothes he wears.

Every new country passes through four stages of transportation: Water; paths and roads; rail; and, finally, the one through which this country is now passing—that of co-ordination and co-operation.

Each of these stages follows as naturally as day follows night; each performs its function satisfactorily considering the condition involved; and each, in the order named, inevitably leads to the other. As soon as a better and cheaper, and more satisfactory, form appears, the less satisfactory, and the more expensive, form fights constantly to prevent the people of the United States from enjoying such transportation, and always on the ground that:

> "The existing forms of transportation fill all our needs, and if you allow competition it will destroy our revenue. We are too important to be destroyed or hurt; we have expanded at the peoples' demand, and it is unfair competition to subsidize some other form of transportation to our disadvantage."

It will be observed that there is no thought anywhere, in this argument, that the people who have "paid and paid" to get transportation, who have allowed one form to supersede another (and always by subsidies), have any legal, moral, or ethical right to patronize that form of transportation which is of most value to them.

The problem confronting the people of the United States is this: They have nearly $1 500 000 000 invested in harbors and navigable waterways, and they have been appropriating annually vast sums of money to continue making navigable streams, in the hope that a cheaper means of transportation will be created whereby the entire country will profit. This vast scheme has only been partly completed, and it will require a comparatively small sum to finish that part of it which has been found to be in accordance with sound economics and sound engineering principles. On its completion the people may look with certainty to the fact that cheaper and better co-ordinated water-rail-motor service will result in a system of transportation superior to any single system, affording every one a real return on the money which he has invested in the form of taxes.

These savings inherent in water transportation will not only be available to those communities fortunately located on navigable streams, because when this co-ordinated system of joint routes and rates is fully developed in accordance with the Denison Act, the shipper in the interior will receive the same savings in cents per hundred pounds through joint rates, as the man who lives near the river. The general principle of determining rates is to apply the water saving between the river ports to the all-rail rate between the points of origin and of destination, and then to establish a joint route, *via* the Federal Barge Lines, by rail and water, or by motor and water, cheaper than the all-rail or all-motor rate. For example, if the all-rail rate on a given commodity from Dubuque, Iowa, to New Orleans, La., were $6.00 per ton, the all-water rate normally would be $4.80 per ton (80% of the all-rail rate), a saving of $1.20. Now, if the all-rail rate from Wightman, Iowa, to New Orleans, for example, were $7.00 per ton, a joint route and rate would be established by rail from Wightman to Dubuque, by water to New Orleans, and the joint rate from Wightman to New Orleans would be $7.00 minus $1.20 (the water saving), which equals $5.80 per ton. The Wightman shipper would thus receive exactly the same savings in cents per hundred pounds as the Dubuque shipper.

This is exactly what the Inland Waterways Corporation proposes to do for every interior shipper; and yet the opponents of waterway development and of the operations of the Federal Barge Lines propose to abandon this vast network of waterways already partly completed, charge it to profit and loss, and admit that the entire policy of 110 years has been a colossal failure, instead of spending the money necessary to complete the system. It is submitted that no business organization, no combination of capital, would abandon any such sum as the $1 500 000 000 already spent to finish seven-eighths of a project which, until completion, is of little value, when by the appropriation of the additional 12½%, a reasonable return on the investment would become available.

In an endeavor to becloud the issue, and by using the same tactics that administrators of canals and highways used against railways in their early pioneering, a concerted attack is being made on the plan of the Federal Barge Lines to utilize streams, on the grounds that this organization is subsidized. It is an agency created by Congress to demonstrate that water transportation is feasible, and economically sound, and it is the only agency available for such demonstration.

Although the Federal Barge Lines have had a net operating income since the inception of the Inland Waterways Corporation, although no money has been appropriated for anything except expansions of the line, although the Corporation could exist and operate and furnish cheap transportation indefinitely on its net operating income, tables such as that in Fig. 10[31] are constantly being published to lead the public to believe that it is being mulcted. It is well to note that the Inland Waterways Corporation has existed only since the latter part of 1924; yet these charges apply to the total expenditures on the waterways on which the Corporation operates, or expects to operate.

---

[31] From *New York Times*, December 28, 1930.

These attempts to prove that there are certain "hidden charges" which the citizen pays when he ships by the Federal Barge Lines, are only efforts to detract the shipper's attention from one important fact; namely, that he is actually getting cheaper transportation by water.

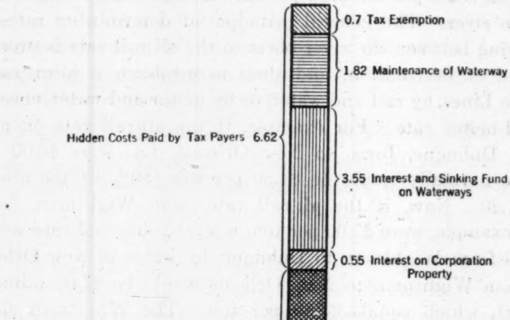


FIG. 10.—"DISCLOSED AND HIDDEN COSTS," IN MILLS PER TON-MILE, OF INLAND WATERWAYS CORPORATION'S OPERATIONS

Certainly the people have been taxed to create and maintain their navigable streams, harbors, lighthouses, etc., to accomplish a certain definite purpose; and since they have been taxed once to create them, why, in order to bolster up a case of "hidden costs", should they be charged again with "tax exemption," "maintenance of waterway," "interest and sinking fund on waterway," and "interest on the Corporation [their own] property?"

The Inland Waterways Corporation is the agent created by Congress to prove that having thus taxed themselves, the people can get that for which they have been taxed, namely, cheaper transportation by water. This is something they had been unable to secure for years, because water transportation has been destroyed by the railroads; and to tax the Corporation, in its pioneering, demonstrative work with a preposterous system of charges which do not exist, and which the people have willed should not exist, is to deny to the people the right to reap the benefit of the investment made.

Assuming, however, that what opponents claim should be charged against water transportation in Fig. 10, are actually things which should be considered, certain errors and discrepancies may be pointed out so that the reader may judge whether or not they make a legitimate case.

The diagram (Fig. 10) was evidently prepared from data contained in the annual report submitted by the writer in 1928. It claims a "hidden cost" to the taxpayer of 0.7 mill per ton-mile on the ground of "tax exemption." The distance from Minneapolis, Minn., to New Orleans is 1 839 miles. According to Fig. 10 the hidden cost, "tax exemption" on a ton of freight transported from St. Louis, Mo., to New Orleans, is $0.8078. Since the freight charged on a ton of grain from St. Louis to New Orleans is $1.60, the "hidden tax exemption" is 50% as great as the cost, if the diagram is correct.

The total corporation "tax exemption," hidden cost, given in this diagram for 1928, would amount to $1 066 741.90; or (since the freight revenue for the year was $6 445 353.70) more than 16% of the total revenue. According to statistics of railways in the United States, published by the Interstate Commerce Commission for 1928, taxes of Class I steam railways were equal to 6.37% of the total operating revenues. Why charge the Inland Waterways Corporation 16 + %, or three times as much as the railways? On the same basis that railroad taxes are calculated, this "hidden cost" would be 0.23 mill per ton-mile.

On the same basis, the "hidden cost," as given by the diagram of "maintenance of waterways," would be $2 773 528.94; "interest and sinking fund," $5 409 905.35; and "interest on corporation property," $838 154.35 (a total of $10 088 330.54), while the total freight revenue received by the Corporation actually amounted to $6 445 353.70.

Why is this item, "maintenance of waterways," charged to the Inland Waterways Corporation? According to a pamphlet issued by the U. S. Department of Commerce there are 200 common carriers, 98 contract carriers, and 187 private carriers, with a total value of $150 000 000, operating on the inland waterways of the United States. In 1928, to which year Fig. 10 evidently referred, the value of the Inland Waterways Corporation was approximately $16 000 000, or 10.66% of the total invested on inland waterways. A proportionate share, therefore (if such charge is legitimate), would be 0.19 mill per ton-mile for maintenance of waterways, instead of 1.82 mills per ton-mile. The interest and sinking fund on waterways (if such charge be legitimate) would be 0.378 mill per ton-mile.

"Interest on the corporation property" is here computed at 5.5% of the actual value at the time. As a matter of fact the Government can get all the money it wants at 4%, so that the interest would amount to only 0.4 mill per ton-mile. Now, add to the actual items, the "disclosed costs" (which included a net income of $327 712.30), and all that Fig. 10 purports to give as hidden costs, in order to obtain the final cost per ton-mile. Then the results are, as follows:

Item	Mills per ton-mile
Tax exemption	0.26
Maintenance of waterway	0.19
Interest on sinking fund, etc.	0.38
Interest on Corporation property	0.4
Cost paid for transportation	4.23
Total	5.46

This may be compared with the 10.85 mills per ton-mile in Fig. 10. In comparison, the actual operating cost of the railroads equals almost exactly 10 mills per ton-mile, without considering any "hidden costs" in the form of land grants to the railways since 1850, amounting to 132 173 224 acres, or of the subsidy of the $40 000 000 per annum received through Pullman surcharges, or other items.

This "land grant" by the Government to the railroads is a continuing subsidy, since in the fiscal year, 1930, there were 62 249.29 acres of land certified or patented on account of railroad grants by the United States to the railroads precisely as they were certified or patented (or granted) in the early days. One cannot be certain exactly what these 132 000 000 acres, more or less, were sold for, but according to an advertisement issued by the Northern Pacific Railroad Company in 1871, the lands granted to the Union Pacific sold for $4.46 per acre; the school lands of Minnesota, $6.30 per acre; and the lands of the Illinois Central grant at $11.00 per acre, and the advertisement quotes an "average of $4.00 per acre." The Government price per acre at this time was given as $2.50, so that, taking this extremely low price, there is a "hidden cost" to the people of the United States of 132 000 000 acres at $2.50 per acre, or $330 000 000, given to the railroads to secure a "right of way," just as the money spent for rivers was given by the people to the people themselves to secure a "river right of way." Furthermore, the rights of way thus given to the railroads by the Government are carried in the value of the railroads, on which the people are requested to pay an additional 5¾% return.

The United States Government spends millions of dollars on harbor improvements, lighthouse service and construction, building jetties, dredging bars, and keeping channels clear and open, in order to aid ocean transportation and make possible the delivery by the railroads of multiplied train loads of freight and passengers. Without this expenditure the seaport terminals of the railroads would be dead ends and would soon become worthless. Of course, the Government is not meeting this great annual outlay specifically for the railroads, but as a means to an end, and to build up, protect, and stimulate the transportation service of the entire country and, also, in the interest of all the people, and it is manifestly a good policy. The people need transportation and they develop it through one form or another of subsidy.

They are entitled to the best form of transportation available at the cheapest possible rate, and if they desire to pay a part of the cost of transportation by highway, or by inland waterways, or by granting to the railways the present existing $40 000 000 subsidy of a surcharge on a Pullman ticket, they have a perfect right so to do. It would appear to be a much better policy on the part of all concerned not to attempt to mislead the people about hidden costs, but to furnish the best and cheapest possible transportation, in fair competition; and that system will come about through proper co-ordination and co-operation of all forms of transportation. It cannot be brought about by crying aloud that one form of transportation will destroy another, or by bickerings, evasions of self-evident truths, or unfair propaganda.

L. D. CORNISH, M. AM. SOC. C. E. (by letter).—The writer wishes to record his appreciation of the very able discussion accorded his paper. Mr. Hadley's argument that inland waterway operation will not lower rail rates because of the fixed differential, is true only with respect to waterway common carriers regulated by the Interstate Commerce Commission. The competition of contract and private carriers on the Ohio and Mississippi Rivers has resulted, during 1930, in certain railroads applying for reduced rates on steel products from the Pittsburgh District. Contract and private carriers unregulated as to rates also constitute a real menace to the development and existence of the waterway common carrier because of their rate-cutting methods.

Professor Sorrell's discussion the writer considers as a conspicuous accomplishment of the main purpose of his paper, namely, to invite or provoke discussions of the subject by trained and experienced students of the transportation problem. His discussion of the various forecasts of future traffic demand and the methods used should be valuable to those interested in that phase of the subject. It appears that the writer's forecast of 870 000 000 000 ton-miles is too large even if it be considered as the shippers' demand on rail, truck, and inland water transportation agencies exclusive of the Great Lakes. This consideration becomes necessary on account of the phenomenal growth of truck traffic since 1925. Truck traffic constitutes a serious menace to the future welfare of the railroads. They must meet this competition by reducing their short-haul and terminal costs and the consequential freight rates and must provide store-door delivery for L. C. L. freight.

Railroad officials and railroad periodicals lay much emphasis on the fact that railroad companies pay property taxes while truck carriers have the tax-free use of highways built and maintained at public expense. It is surprising, indeed, that they apparently overlook the tax on gasoline and its effect on truck operation costs. Every State in the Union taxes gasoline for motor vehicle consumption. The tax ranges from 2 to 7 cents per gal., and probably averages about 4 cents. Heavy motor trucks consume about 0.06 gal. per cargo ton-mile, and hence their gasoline tax for use of highways is about 2.4 mills per ton-mile, which is 3 times the property tax and about 1.6 times the maintenance cost paid by the railroad companies.

The suggestion of Professor Sorrell of a method of arriving at the probable social cost of a ton-mile of waterway transportation and the net social saving is very interesting. The writer feels indebted to Professor Sorrell for his splendid contribution to the subject and especially for his constructive criticism and discussion of the writer's paper.

Professor Kittredge charges the writer with two errors of major importance. First, that "figures which cover the total cost of railway transportation have been used in comparison with other figures which include only a part of the cost of waterway transportation".

This should not be charged as an error for the reason that the writer made no pretension of drawing a comparison between the respective total costs, including past expenditures, for either railways or waterways. To have attempted that would have been futile and the results of only academic interest. Why be concerned about the economic wisdom of past expenditures, or interest charges on them? The writer's comparisons and conclusions refer only to the Mississippi Valley systems. Federal appropriations therefor were provided by taxes, not bond issues, and the public is not paying interest on them. If one must consider all this water (which is "over the dam" with respect to waterways), why not then, give similar consideration to the millions and possibly billions of dollars of capital which has been lost by investors in the re-organization of bankrupt railroads?

The writer specifically limited his discussion to future costs for railroad extensions and to the completion of Mississippi Valley waterway improvements which, at that time, had been approved and authorized by Congress.

The second error charged by Professor Kittredge is to the effect that the writer claimed as an economic advantage of the waterways an annual amount of $73 600 000 to cover interest on railway capital expenditures saved by the completion and utilization of the Mississippi Valley waterways.

The writer realized that the amount of this claimed advantage was highly questionable and so pointed out in his paper. Professor Kittredge and others called this to the attention of the writer previous to the publication of the paper. All these critics admitted there was justification for some claim of this kind, but did not suggest a method of evaluating it. After due consideration the writer decided to let the claim stand as originally made in the hope that the subject would be clarified in the discussions and that it would serve as a target to invite discussion of the entire paper, particularly by railroad officials and economists.

The Traffic World, of which Mr. Palmer is Editor and Manager, is severely critical of the development of inland waterway transportation. While undoubtedly he is well qualified to discuss or challenge statements made by the writer and others who prepared the papers of this Symposium, Mr. Palmer has not (in so far as the writer can determine) chosen to do so. His discussion is apparently only a presentation of his personal opinion on a few of the general phases of the subject.

Mr. Palmer admits that "if it is found that water transportation fills a need that can be supplied in accordance with sound and reasonable economy it should, of course, be supplied", but he refuses to accord to water transportation the right to any form of paternalism or public support during its development period, although the preliminary development of railroads was due to this same form of paternalism.

The discussion by Mr. Brent is confined to the writer's paper and is principally devoted to showing its conservatism. This is, indeed, gratifying to the writer, particularly in view of Mr. Brent's broad knowledge of the subject gained from many years' experience with the Federal Barge Line and other transportation agencies, both water and rail.

It should be noted, however, that Mr. Brent discusses rail costs and traffic densities on the basis of miles of road, whereas the writer discusses them and presents figures on the basis of miles of track. By the latter method costs and efficiency are reduced to a basis common to all railroads, whether of single or multiple-track construction. The Pittsburgh and Lake Erie Railroad mentioned by Mr. Brent has 231.46 miles of road, but it has 1 200 miles of main track. Hence, while its road-mile traffic density (10 700 000 ton-miles) is the greatest, its track-mile density—the true measure of efficiency—is 2 060 000 ton-miles. The Virginian Railroad has the greatest track-mile density

(3 400 000 ton-miles) of any railroad in the United States. Mr. Brent's statements and arguments based on miles of road are correct and would be equally applicable and forceful if reduced to a miles-of-track basis.

The railroad point of view in this controversy has been so seldom presented, except in generalities, that the discussion by Mr. Dunn, who is Editor of *The Railway Age*, stands out as a most valuable contribution to this Symposium. Mr. Dunn presents his data and arguments and conclusions in sufficient detail so as to invite specific definite argument in rebuttal. His views, however, are based upon certain fundamentals which are really the points in controversy. He denies the economic soundness of any form of public subsidy to waterways which is not fully reflected in the cost of water transportation. Recurring annual subsidies should be so reflected, but why should interest on accumulated construction costs provided by the public for channel improvement be added to operating costs when the public is not being taxed for them? There is no more reason for adding such a theoretical interest charge to waterway operation costs than there would be in adding to rail rates a fair return on the billions of dollars of capital which has been lost by investors in the re-organization of bankrupt railroads.

While Mr. Dunn separates the cost of inland waterway transportation into two parts, namely, subsidy cost and the cost of the actual movement of freight, he insists on combining the two for comparison with rail rates, in order to show that the cost of water transportation is greater than rail freight rates. It is true that the public will "foot the bill", but as National or sectional prosperity depends more on the actual moving of the freight than it does on the amount of the freight bill, it is evident that it is the freight rate which must be the governing factor, and that this rate must not be greater than the traffic will bear. For two decades the agricultural and manufacturing industries of the Middle West have been slowly "strangulating" under freight rates which the traffic could not bear, at the same time permitting the industries to prosper. The public in the Middle West at last has realized this condition and believes that in inland waterway transportation lies the solution of the problem. Waterways are Nature's God-given free highways to Man and the laws of all great nations, including those of this country, recognize them as such. Therefore, it is an obligation of the Federal Government to maintain such of these channels as may be used to the economic benefit of the sections where they exist. The extent to which such channels will be used will depend upon the terminal and operating costs of waterway movement plus the profit required to secure the necessary capital. If the resulting freight rates attract commerce in considerable volume, the community will benefit. Some railroads may suffer for a time; but such incidents are only passing events in the march of progress, which cannot be stopped in a nation of free people. A far greater menace to present railway operations exists in the remarkable growth of truck and bus traffic. The haulage costs of truck traffic greatly exceed rail costs. This is a basic economic advantage possessed by the railroads, and hence they should be able to recapture that business.

From his viewpoint of the subsidy question Mr. Dunn correctly shows the present status of costs on the New York Barge Canal, although better authorities than the one he quotes have repeatedly asserted hidden public benefits to an amount greatly in excess of the "hidden" cost mentioned by Mr. Dunn.

The part of Mr. Dunn's discussion relative to the Ohio River System costs is answered in the discussions by Colonel Kutz and Mr. Hobbs, both of whom (as U. S. War Department officials) possessed authoritative knowledge of traffic and costs on that System.

Mr. Dunn's criticism of the writer's forecast of rail ton-mileage for 1950 is well founded. It is probably from 10 to 15% too high. This forecast (Curve *B*, Fig. 5) was derived in 1925 from railway records to 1924. Truck traffic was not considered because its importance was not realized. Authorities now state that the present truck traffic is about 10% as great as rail traffic. If this is true it would indicate that truck traffic in 1929 amounted to 45 000 000 000 ton-miles. Adding this to the rail traffic gives a combined total of 495 000 000 000 ton-miles which is exactly 1% less than the ton-miles shown for 1929 on the forecast curve, *B*, in Fig. 5 of the writer's paper. It is evident, therefore, that the forecast portion of Curve *B* should be considered as indicating the combined public demand for rail and truck traffic inclusive of such water traffic as may be diverted from those two agencies of transportation by the increase over the present volume of inland waterway transportation. Such a correction would require a downward revision in all the writer's forecasts of future railway development expenditures. This revision would not be nearly as extensive as indicated by Mr. Dunn, and to make it now would serve no useful purpose.

The writer is too firm a believer in the industrial future of the United States to accept Mr. Dunn's forecast of 575 000 000 000 ton-miles for rail traffic alone. To accept such a figure requires one to agree that the future rate of industrial progress of this nation not only will not equal the rate of the past forty years, but will progressively decline. If no greater traffic than this may be expected, then, as noted by Mr. Dunn, the writer's analysis shows that no additional trackage or equipment will be required. However, while equipment capacity has fallen off, track construction has continued, and capital investments of the Class I roads since 1924 have increased at a rate 50% in excess of the writer's estimate for freight traffic growth.

As a closure to this discussion, attention is invited to the publication in October, 1930, by the "Bureau of Railway Economics", of a 238-page report entitled "An Economic Survey of Inland Waterway Transportation in the United States". This work contains all authoritative facts and references on the subject and their analyses from the railroad point of view, as to present and past conditions only.

Certain cost data from this report have been widely quoted as proof that the cost of water transportation is greater than existing rail rates and that, therefore, waterway improvement is uneconomic. The report contains (pages 193 to 198) a summarized review of Mississippi and Ohio River costs for the year 1928. The facts are probably correct, but if intelligently and fairly used they offer all the proof required to demonstrate the economic benefit of water transportation on the Mississippi River System. It is needless to say that the so-called subsidy items of channel maintenance and annual interest on accumulated construction capital cost for the past 100 years are included. The writer offers no protest at the inclusion of these items because the waterway traffic can stand it. The tabulations show the cost of these items per ton-mile of traffic in 1928, but the ton-mileage of that year is only a small fraction of the waterway capacity or the traffic reasonably to be expected within the next twenty years.

The ton-mile costs shown in Column (3) of Table 10 are exactly as they appear in the report (page 196) but Column (4) shows the costs based on a traffic of 45 000 000 000 ton-miles, and the total capital cost is assumed at $270 000 000 to cover the cost of completion of the project. Annual maintenance is increased to $6 000 000 for the same reason. Column (3) clearly shows the economic soundness of water transportation provided it can be developed to a satisfactory volume.

TABLE 10.—STUDY OF TON-MILE COSTS, MISSISSIPPI RIVER

Item (1)	No. (2)	1928 (3)	1938 to 1950 (4)
Annual capital cost per ton-mile	1	1.84*	0.24†
Maintenance cost per ton-mile	2	1.61‡	0.13§
Freight-carrying cost per ton-mile	3	4.00	4.00
Sub-total	4	7.45	4.37
Allowance of 50% for circuity	5	3.72	2.18
Total cost per ton-mile	6	11.17	6.55
Weighted average freight receipts of eight railroads near the Mississippi River	7	10.09	10.09
Total, in ton-miles	8	2 951 000 000	45 000 000 000

* 4% on $135 000 000. † 4% on $270 000 000. ‡ 4% on $4 753 000. § 4% on $6 000 000.

Referring only to the 1928 values in Table 10, the report (page 196), states: "The rail costs average 10.09 mills per ton-mile; the estimated *minimum* transportation costs on the Mississippi River average 11.17 mills per equivalent ton-mile, after allowance is made for the greater circuity of the water route."

Attention is invited to the unwarranted use of the word, "minimum", in the foregoing statement.

Inasmuch as the Bureau of Railway Economics has included in its waterway costs for 1928 all items which it considers necessary to conform to sound accounting practice, the total thus derived should be reduced by the 20% freight-rate differential saved on water shipments before a comparison is made with rail rates. The freight-carrying cost of 4 mills per ton-mile increased 50% on account of circuity or greater length of water haul over rail haul between ports becomes 6 mills. However, since the shipper, in paying this amount, saves 20% of the rail rate, his saving becomes 25% of 6 mills, or 1.5 mills per ton-mile. Subtracting 1.5 from 11.17 and 6.55, respectively, leaves 9.67 mills per ton-mile in 1928 and 5.05 mills per ton-mile by 1950, at which time (as indicated on Fig. 9) traffic will equal 45 000 000 000 ton-miles per annum on the Mississippi River alone. This indicated public cost of 5.05 mills for future traffic is the figure which should be compared with rail rates of 10.09 mills per ton-mile to determine the relative public economy of the two methods of transportation.

Thus, it is shown that, by the railroad method of accounting, waterway transportation costs to the public on the Mississippi River in 1928 were slightly less than average rail rates, and that if and when water traffic equals 45 000 000 000 ton-miles per annum, the cost will be 5 mills per ton-mile less than present average rail rates.

The Bureau report (page 198) shows, in the same way, that water costs on the Ohio River in 1928 were 12.36 mills per ton-mile and that comparable average rail rates were 8.83 mills. The water costs covering capital and maintenance are based on 2 702 416 316 ton-miles of traffic in 1928. Fig. 9 indicates that by 1945 Ohio River traffic will be five times the present traffic, in which event the capital and maintenance charges would be one-fifth of the amounts shown by the Bureau report for 1928. By making this adjustment the indicated water cost for 1945 becomes 6.91 mills per ton-mile, or 1.92 mills less than present average rail rates.

Numerous railroad officials and economists in public addresses (many of which have been reproduced in the public press) and railway periodicals, supplemented by editorial agreement therewith, have repeatedly asserted that, if inland waterway transportation on the Mississippi River had to bear the cost of the construction and maintenance of the channel as railroads do with respect to this right of way, the cost of water transportation would be shown to be greater than the cost by rail. Their methods of accounting for capital and maintenance of channel costs are not in accord with the National policy. However, if their method is accepted as economically sound, their assertions are reasonably correct with respect to present-day traffic, but very incorrect if water traffic develops in large volume. Economic justification, therefore, depends not upon present-day costs, but upon the development of a large volume of water traffic which will reduce these overhead, or so-called subsidy, items to almost negligible figures.

E. A. Hadley, M. Am. Soc. C. E. (by letter).—The several discussions of the writer's paper have been given careful consideration. The operating statistics and capital expenditures made by the railroads are readily available to any one interested in the subject, from the reports which are required to be made to the Interstate Commerce Commission; and all such statistics and reports are made on a uniform basis in a manner prescribed by the Commission.

On the other hand, information and statistics regarding waterway development and operation must be gathered with great effort from a large number of official reports of various departments of the Government. Many of these reports are issued by the Chief of Engineers of the United States Army and others appear in the reports of the Inland Waterways Corporation, a Government organization. The annual reports of the Chief of Engineers consist of bound volumes aggregating from 3 000 to 6 000 pages, from which it is no small task to assemble data pertinent to the subject under discussion.

The Bureau of Railway Economics, which has been established by the railways of the United States for the scientific study of transportation problems, issued a volume, designated as Special Series No. 56, and dated Washington, D. C., October 15, 1930, entitled: "An Economic Survey of Inland Waterway Transportation in the United States." The information and statistics given in the volume are as nearly correct as it is possible to have them, based upon the official and unofficial sources of information from which it has been compiled. This volume furnished a considerable portion of the foundation for the discussion in the writer's paper.

A joint report[32] of the U. S. Corps of Engineers and the U. S. Shipping Board issued in 1927 has laid down the principle that when the cost of water transportation is to be compared with the corresponding cost of rail transportation, the former should include interest charges on the capital investment in the waterway improvements plus the annual cost of operation and maintenance. Using these basic principles, and applying them to available statistics regarding the Mississippi River, the Ohio River, and the New York State Barge Canal, the following situation is developed:

> (1) Transportation costs on the Mississippi River are not less than 11.17 mills per ton-mile, with allowance made for circuitous river channels; on eight railroads which, in general, parallel that river, the average inclusive freight charge per ton-mile is 10.09 mills.
>
> (2) Transportation costs on the Ohio River, determined in a similar manner, are not less than 12.36 mills per ton-mile; on seven railways which, in general, parallel that river, the average inclusive freight charge per ton-mile is 8.83 mills.
>
> (3) Transportation costs on the New York Barge Canal are computed at 19.41 mills per ton-mile; on the railways of the Eastern District the average inclusive freight charge per ton-mile is 10.90 mills.

It must be remembered that the financial statements issued by the Inland Waterways Corporation do not make any allowance for return on the capital stock issued by it to the Government. The expenses of the Corporation do not include any provision for taxes because the Government pays no taxes to itself. These items must be considered a charge against revenue before any private investor would judge the operations of the Barge Line to have carried out the purpose for which it was established, namely, to demonstrate whether or not the service can be carried on by private capital at a profit.

Because of lack of statistics it is not possible to compare the operations of private river carriers and those of the Government Barge Line with respect to their profitableness. Even if the private carriers use free-of-charge, improved waterways made possible by expenditures of Government funds, they do pay taxes and may be presumed to earn a return on their investment or they could not remain in business. The same condition is true of those companies which utilize the improved waterways for hauling their own raw materials and products in that they use the waterways free of tolls and find the water transportation a more or less profitable extension of their plant operations.

---

[32] "Transportation on the Ohio River System" (Interim Rept.), 1927, p. 17.

The statement is repeatedly made that the present means of transportation will not suffice to meet the future growth of traffic incident to the growth of the country in population and wealth. President Roosevelt in his letter creating the Inland Waterways Commission, in 1907, wrote:[33]

> "It is common knowledge that the railroads of the United States are no longer able to move crops and manufactures rapidly enough to secure the prompt transaction of the business of the nation, and there is small prospect of immediate relief. * * * There is reason to doubt whether any development of the railroads possible in the near future will suffice to keep transportation abreast of production."

President Roosevelt had before him the relatively large rail-traffic totals for 1906 when he wrote this letter. Despite his predictions, however, the rail traffic of 1929 was more than double that of 1906, and this increased traffic was moved promptly without congestion and without delay to shippers or receivers.

In its 1908 Annual Report, the Inland Waterways Commission, stated:[34] "It is clear that at seasons recurring with increasing frequency they (the railways) are unable to keep pace with production or to meet the requirements of transportation."

The National Waterways Commission in 1912 stated[35] that the rapid expansion of traffic between 1904 and 1907 had exhausted the surplus capacity of the railroads and added: "With the next period of expansion there is strong probability of a worse congestion of traffic than occurred before."

While he was Secretary of Commerce, President Hoover made addresses throughout the country advocating the consolidation of the scattered sections of improved waterways. In this connection he made the statement that the growth of population within twenty-five years would produce such an increase of traffic that the railways would be unable to meet it. Using past history as a basis for forecast of the future, the railways will undoubtedly keep ahead of the growing demands of traffic upon them. For several years no traffic congestion has occurred and reports of the Department of Commerce have emphasized the remarkable improvement of the transportation service rendered by the railway industry.

The construction and operation of any system of transportation involves large expenditures of money and it may be briefly considered that the principal items of expenditure are, as follows:

(1) The cost of construction of the system itself, which usually runs into hundreds of millions of dollars for any large project. In the case of a railroad its cost covers the right of way, tracks, and other permanent structures. In the case of an inland waterway, it covers the cost of construction of the canal or improvement of the river, with locks, dams, and other appurtenant structures.

(2) The annual charge for capital; that is, the annual interest charge on the money invested in the construction or improvement.

(3) The cost of necessary equipment, whether in the form of railroad locomotives and rolling stock, or of ships and barges.

---

[33] Preliminary Rept., Inland Waterways Comm., 1908, p. 16.

[34] *Loc. cit.*, 1908, p. 19.

[35] Final Rept., National Waterways Comm., 1912, p. 575.

(4) The annual carrying charge for capital invested in equipment.

(5) The cost of maintenance, both of the transportation route (railroad right of way and tracks, or river and canal), and of the equipment operated thereon.

(6) Proper charges for depreciation on plant and equipment.

(7) Necessary payments of insurance premiums and taxes.

(8) The direct cost of the transportation service itself, including wages, cost of fuel, materials, and supplies, and the cost of supervision and general expense.

Unless all these items are considered and accounted for, no fair comparison of the relative costs of the two forms of transportation can be made.

The paper presented by Mr. Cornish as a part of the Symposium makes an attempt at forecasting the annual freight transportation demand of the United States by 1950, together with an estimate of additional railroad facilities required to provide for proper handling of that portion of the increase which would go to the railroads.

In discussion, Professor Sorrell sets forth in a most convincing manner the fallacy of attempting to forecast the probable freight traffic to be carried by the railroads in 1950. Furthermore, he states the results of similar forecasts brought down to the year 1943 showing such wide variances that he reaches the conclusion that "it is unsafe to assume that the trend for the next twenty-five years will be on the same basis as that for the last forty."

In his comparison of costs of transportation between the railroads and the waterways, Mr. Cornish overlooks the fact that the present average ton-mile earnings of the railroad include return on the investment and taxes, and he has also used the arbitrary differential of 20% in water rates under the rail rates as his measure of the saving in transportation costs by use of the waterways.

Mr. Cornish has further disregarded capital outlays already made for river improvements and terminals by use of public funds, except those terminals used by the Federal Barge Line. He also assumes that the present average freight receipts of the railroads of about 11 mills per ton-mile will prevail in 1950; however, should the increase in traffic on the railroads develop to the point estimated by him, it is probable that freight rates could be reduced materially, which would lessen the savings he estimates due to waterway improvements.

On the other hand, if all the figures going to make up the cost of waterways transportation are considered, the estimated savings on this account would largely disappear, particularly if the gain in waterways traffic did not reach the total of his estimate.

Many of the same fallacies in reasoning appear in the discussions by Colonel Kutz, and Mr. Hobbs. Mr. Brent comments somewhat on the paper by Mr. Cornish and toward the end of his discussion makes the following statement:

> "The day of cheap railroad transportation is past. Sweeping increases in rates are being made in all important territories to bring railroad revenues to the mark fixed upon as necessary to support and expand the plant."

In 1922, the Interstate Commerce Commission ordered a 10% general reduction of freight rates in the belief that this would give "fuller assurance to the carriers of realizing the fair return contemplated by the law." Since that date the Commission has granted no "sweeping increases in rates", but on the contrary there has been a continual paring downward of rates so that the present per ton-mile earnings of the railroads are less than they were following the reduction made in 1922, and there appears to be no indication on the part of the Commission of allowing any general increase in rates.

In the last analysis it appears that most of the proponents of waterways transportation give consideration to only a part of the costs of such service in making their comparisons, the remainder being borne by the Federal Government, or other public authority, while the railroad figures include all the elements of such costs and are borne directly by the railroads themselves. The total cost of each form of transportation should constitute the only basis for comparison.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

TRANSACTIONS

Paper No. 1780

FREQUENCY AND INTENSITY OF EXCESSIVE RAINFALLS AT BOSTON, MASSACHUSETTS[1]

BY CHARLES W. SHERMAN,[2] M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. W. W. HORNER, C. S. JARVIS, MILTON F. WAGNITZ AND LEWIS C. WILCOXEN, AND CHARLES W. SHERMAN.

SYNOPSIS

This paper presents the results of a study of the records of the recording rain gauge at Chestnut Hill Reservoir in Boston, Mass., covering a term of 50 years, and durations of heavy rain up to 72 hours. Curves of intensity of precipitation, corresponding to any period of time up to 72 hours, for various frequencies of occurrence are derived and discussed.

The form of curve showing the relation between the duration and the (average) intensity of precipitation is discussed, and the possibility of the application to other localities of the general formula herein derived, with the coefficients and exponents suitably modified, is suggested.

INTENSITY OF RAINFALL

The rate at which precipitation occurs during heavy storms has been recognized for many years as the most significant of the information required in the design of storm-water drains and combined sewers. At first, the hourly rate, often taken as the presumable maximum for the locality, was commonly used, irrespective of the magnitude of the area to be drained. Later, it was recognized that the "time of concentration" for the particular area represented the period for which the rate of precipitation was desired; and it was noted that there was a relation between the length of the period and the corresponding intensity of precipitation.

For some time it was held that only periods of uniform intensity of precipitation were of significance in estimating storm-water run-off from rates of rainfall. In particular, the late Emil Kuichling, M. Am. Soc. C. E.,[3] emphasized the fact that if the rain fell at varying rates during the period of concentration, it was practically impossible to make trustworthy estimates of run-off from the rainfall record. If this criterion were rigidly applied, there would be comparatively few records of heavy rain of more than 20 or 30 min. duration which could be used; yet it is obvious that the effect of storms of variable intensity in producing run-off must usually be as great as, and sometimes greater than, that of a storm of uniform intensity and the same total precipitation in the particular period of time. Therefore, average intensities have been accepted as significant, and are utilized in the same way as though the precipitation fell at a uniform rate throughout the period under consideration; although it is recognized that the rate of rainfall was substantially uniform throughout the period in comparatively few cases where the duration exceeded 30 min.

---

[1] Published in April, 1930, *Proceedings*.

[2] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

Recording Rain Gauge at Chestnut Hill Reservoir

This gauge was established by the late Desmond FitzGerald, Past-President, Am. Soc. C. E.,[4] in 1878, and there are now records from it covering a period of fifty years. It is maintained by the Metropolitan District Commission. A description of the gauge and its exposure is contained in a paper by Harrison P. Eddy, M. Am. Soc. C. E., entitled "Maximum Rates of Precipitation at Boston for Various Frequencies of Occurrence".[5]

Effect of Elevation of Gauge.—The collector of this gauge is on the roof of a building, about 25 ft. above the ground. An ordinary rain gauge is maintained at ground level, about 800 ft. from the present location of the recording gauge and about 450 ft. from the position which it occupied previous to 1898. In general, the total precipitation in any storm as shown by the elevated gauge was somewhat less than that collected in the ground gauge, although there were a few cases in which the former showed the larger amount.

The ground gauge has been assumed to be correct in all cases, and the records of the recording gauge have been adjusted by applying for each storm, a factor obtained by dividing the total collection, as shown by the ground gauge, by that indicated by the recording gauge. This factor has been applied to all periods of a storm, as there is no basis for modifying it. In value, it varied from 0.95 to 1.56; for the highest intensities, its value ranged from 1.03 to 1.16

Comparison with U. S. Weather Bureau Gauge.—In Mr. Eddy's paper referred to previously, the records of the gauge at the Chestnut Hill Reservoir, for the 38-year period then available, were analyzed and also those of the U. S. Weather Bureau gauge at Boston for 26 years. The distance between the two is about 5 miles. Although there were differences in the results obtained it was concluded that, on the whole, they were as nearly the same as could be expected; and as the records obtained at Chestnut Hill were adjusted by comparison with those of a ground gauge, while no such adjustment of the Weather Bureau gauge was possible, the former were thought to be the better.

---

[3] "The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts," *Transactions*, Am. Soc. C. E., Vol. XX (1889), p. 1.

[4] "Maximum Rates of Rainfall," by Desmond FitzGerald, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XXI (1889), p. 93.

[5] *Journal*, Boston Soc. of Civ. Engrs., 1920, Vol. VII, p. 47.

In this paper, no comparison with the records of the U. S. Weather Bureau gauge is attempted.

Duration of Intense Rainfall.—In earlier analyses of records of intense rainfalls, comparatively little attention was paid to storms lasting more than 2 hours. In the cases in which longer periods were considered, it appears probable that but few of the significant storms were considered.

In view of the greater extent to which storm flow is now being estimated from rainfall, for periods of concentration as long as 2 or 3 hours, or even more, and of the recent application of this method in estimating flood flows of streams for which the period of concentration may be 3 days, or more, particular attention was paid in this analysis to heavy rains covering the longer periods. It was found that, in Mr. Eddy's paper, a number of storms which in reality lasted 3 hours, or more, at high rates were actually included in the study only for periods of 80 min., or less.

All storms which showed a precipitation of more than 1 in. in 24 hours, 2 in. in 48 hours, or 3 in. in total precipitation, were also studied in detail, and rates of precipitation for longer periods up to 72 hours were computed and utilized in the present analysis.

"Extended Duration" Principle.—As a logical extension of the use of average rates of precipitation, all storms in which the total precipitation was sufficient to show significant average rates for periods longer than the actual duration of the rainfall were considered as though they had continued for the longer times. Thus, a storm which yielded a total precipitation of 1.50 in. in 50 min., had an average intensity of 1.80 in. per hour for that period, of 1.50 in. per hour for 60 min., and of 1.13 in. per hour for 80 min., all of which might be of significance in the analysis.

In tabulating the 50 highest intensities for each duration from 5 to 180 min., only 10 of the 650 figures were supplied in this way. In the case of long-time storms, however, a considerable number were tabulated for longer periods than the actual durations of the storms.

Analysis of Records.—The quantities of precipitation, adjusted to agree with the ground gauge, and the corresponding intensities for periods of 5, 10, 15, 20, 25, 30, 45, 60, 80, 100, 120, 150, and 180 min., were tabulated for the significant storms, so as to obtain the fifty greatest intensities for each of these periods, utilizing the "extended duration" principle wherever applicable. The records of long storms were analyzed by hours, to ascertain the greatest intensities for periods of 4, 5, 6, 8, 10, 12, 15, 18, 21, 24, 30, 36, 42, 48, 54, 60, 66, and 72 hours, so as to obtain at least the highest ten intensities for each of these periods.

FREQUENCY

If the 50-year record were a fair representation of the average 50-year period at Boston, then the greatest intensity for each of the periods should be that which is likely to recur once in 50 years, or that which is likely to have a frequency of 50 years. The points of second magnitude represent intensities which might be expected to be equalled or exceeded twice in 50 years, or to have a frequency of 25 years; and the points of the fifth magnitude would have a frequency of 10 years.

It is reasonable to suppose, however, that some of the maximum points may be high enough so that similar intensities will not recur for 100, or even 500 years; the 500-year maximum must occur within some 50-year period, and it may have been in that studied. It is probable also that some of the maximum points may be lower than they would be for an average frequency of 50 years, if the record had covered a longer term, such as 200 years. That is, the curve showing the relation of intensity to duration for a frequency of 50 years can not be very satisfactorily determined from the maximum points of a 50-year record. The 25-year curve is much more definitely fixed, because the record covers two full periods of 25 years; and curves of greater frequency are still more definitely determined.

FORM OF INTENSITY-DURATION CURVE

Intensity-duration curves for the several frequencies which might be expected to be fairly reliable, namely, those occurring three times in 50 years, and oftener, showed that the curves for all frequencies were of the same general form, although there were several places where the smooth curves were at some distance from the plotted points. The similarity in form of curve suggested that a single form of equation, with suitable modification of the constants, might be applicable to intensity-duration curves for all frequencies. Plotting on logarithmic cross-section paper with intensities as ordinates and durations as abscissas resulted in nearly parallel lines of slight curvature, which were convex upward. Then, assume an equation of the form:

$$i = \frac{K}{(t + b)^d} \qquad \text{(1)}$$

in which, i represents intensity of precipitation, in inches per hour; t, the period of time, in minutes; and K, b, and d are constants; it was found by trial that a value of $b = 7.0$ was reasonably applicable to the points of each set (that is, for a particular frequency). Values of i were then plotted as ordinates against $t + 7$ as abscissas, and it was found that all the points of each magnitude, from the third to the fiftieth, would lie very nearly on a straight line, while those for the first and second magnitudes were reasonably distributed about such lines. They were very nearly parallel; and if d were taken as 0.7 in each case, the resulting parallel lines were satisfactory as representing the whole series of observations. The general formula for relation between intensity of precipitation and length of period is, therefore,

$$i = \frac{K}{(t + 7)^{0.7}} \qquad \text{(2)}$$

Value of Coefficient, K.—Values of K corresponding to the third, fifth, tenth, twenty-fifth, and fiftieth magnitudes have been taken from a diagram upon which the lines representing Equation (2) were drawn, with the following results:

Frequency. in years.	Values of K
1	16.0
2	20.0
5	24.8
10	30.5
16⅔	34.6

Values for frequencies of 25 and 50 years were not obtained by drawing intensity-duration curves, since the points departed so much from any straight lines (on logarithmic paper) that the positions of the most probable lines could not be fixed with satisfaction, at least on the basis of the study of precipitations of durations up to 180 min. It appears, however, that the relation between values of K as listed herein can be expressed satisfactorily by the equation:

$$K = 16\ F^{0.27} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which, F represents frequency, in years. The resulting values of K, compared with those taken from the diagram, are shown in Table 1.

TABLE 1.—VALUES OF COEFFICIENT AND RATIOS OF INTENSITY CORRESPONDING TO VARIOUS FREQUENCIES OF INTENSE RAINFALL.

Frequency, in years.	VALUES OF K.		RATIO OF INTENSITY TO THAT FOR:	
	Computed.	From diagram.	1-year frequency.	10-year frequency.
1	16.0	16.0	1.00	0.54
2	19.3	20.0	1.21	0.65
5	24.7	24.8	1.54	0.83
10	29.8	30.5	1.86	1.00
15	33.2	....	2.08	1.11
16⅔	34.0	34.6	....	....
25	38.1	....	2.38	1.28
50	45.9	....	2.87	1.54
100	55.5	....	3.47	1.86

There is doubt as to the propriety of applying the computation to a frequency of 100 years, but it is interesting to note the results secured and compare them with the greatest figures obtained from the records; and they are probably of significance as giving some indication of what may occur sometime in the way of phenomenal precipitation.

Relative Intensities for Various Frequencies.—The relationship shown by Equation (3) indicates that the intensity to be expected for any frequency of recurrence bears a direct relation to that for any other frequency; some of the ratios are arranged for comparison in Table 1.

COMPARISON BETWEEN COMPUTED AND OBSERVED VALUES

Fig. 1 shows graphically the relation between the length of period (duration) and the intensity obtained from the records of the storms of first, second, third, fifth, and tenth magnitudes for all periods up to 72 hours, and of the twenty-fifth and fiftieth magnitudes for periods up to 3 hours. These magnitudes are indicated by numerals beside the observed points. Fig. 1 also shows the curves derived for frequencies of 100, 50, 25, 16⅔, 10, 5, 2, and 1 years. The use of logarithmic cross-section paper makes it possible to bring all this information within the limits of a single chart.

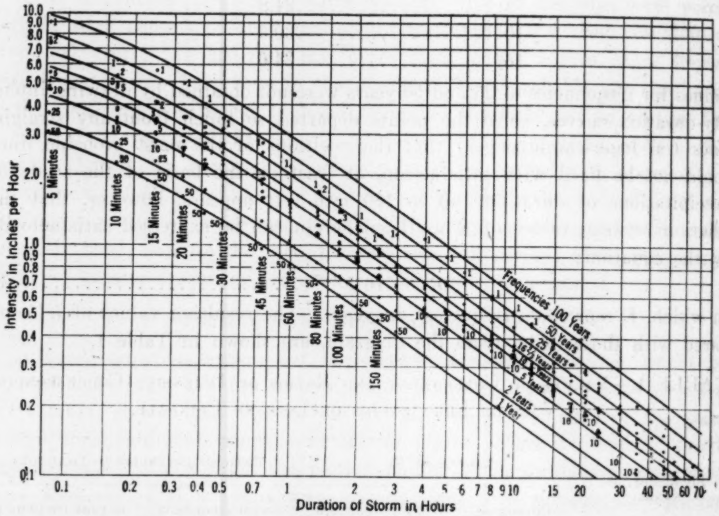


FIG. 1.—INTENSITY OF PRECIPITATION AT BOSTON, MASS., FOR VARIOUS DURATIONS AND FREQUENCIES.

The comparisons are believed to justify the conclusions that Equations (2) and (3) do represent satisfactorily the conditions existing at Boston for frequencies up to 50 years and durations up to 72 hours. There does not seem to be any record of a really excessive storm lasting more than 3 days, and it seems likely that such an occurrence, like total rainfalls in excess of 10 in. in a single storm, will be of great rarity.

Comparison with Previous Studies.—Some of the most significant of the earlier records observed at this station were the basis of another paper by the writer,[6] but the question of frequency was not discussed. Few if any engineers at that time had definite ideas on the relative frequency of rainfalls of high intensity, and records of rates of precipitation were insufficient to warrant any positive conclusion on that matter. A general form of equation was not developed, although two curves were suggested, as follows:

For maximum intensities,[7]

$$i = \frac{38.64}{t^{0.687}} \qquad (4)$$

and for use as a basis for designs,[8]

$$i = \frac{25.12}{t^{0.687}} \qquad (5)$$

---

[6] "Maximum Rates of Rainfall at Boston," by Charles W. Sherman, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIV (1905), p. 173.

[7] *Loc. cit.*, p. 178.

[8] *Loc. cit.*, p. 179.

It is interesting to note that the curves represented by Equations (4) and (5) compare reasonably well with those derived herein for 50-year and 10-year frequencies, respectively.

Application of Formulas to Other Localities

It seems logical to assume that formulas of the same kind may be applicable in other localities. The writer has not found opportunity to make a similar analysis of other rainfall records; but a study made in 1920[9] resulted in the conclusion that the curve of rainfall intensities for the 15-year frequency at Boston, as given by Mr. Eddy[10] was a good one for a 5-year frequency at Springfield, Mass.; and that, in general, rainfall curves for Springfield were nearly like those which had been published for New York City. This indicates the possibility that suitable variations in the values of K may be all that is necessary to make the formula herein derived applicable to a considerable area, such as New England and New York. In other localities, variations in one or both of the constants, b and d, in Equation (1) probably will be required.

Occurrence of Excessive Rates During Storms

In fixing run-off coefficients for use in computing storm flow from rainfall, some engineers use the so-called "zone principle" according to which the area is assumed to be divided into zones on which the rain has been falling for different lengths of time. The assumption is usually made that the rate of precipitation is greatest at the beginning of the storm, and, therefore, that smaller coefficients of run-off can be made with higher, than with lower, rates of precipitation. Comparatively little definite information as to the correctness of this assumption is available, and, therefore, this study was extended to determine the significant data as to the time, with reference to the beginning of the storm, when the excessive precipitation actually occurred.

The records of the ten highest intensities, having durations from 15 to 180 min., were examined to ascertain the quantity of rain falling in a period of equal duration and immediately preceding that in which the intense rainfall occurred. The results are shown in Table 2. They indicate that the assumption of a substantially dry ground surface at the beginning of the intense precipitation is reasonably correct; although there have been cases of heavy rains (not those of greatest average intensity), in which the quantity falling before the intense precipitation began was considerable. The latter condition might result in rates of run-off substantially as high as those from more intense rain falling on practically dry ground.

Storms of Long Duration

It was noted previously that the study of long-duration storms was undertaken because it was believed that any formula representing the relation between intensity of precipitation and duration of precipitation should give

---

[9] "Excessive Rainfall Frequencies at Springfield, Mass," by Charles W. Sherman, M. Am. Soc. C. E., *Engineering News-Record*, September 2, 1920, Vol. 85, p. 445.

[10] *Journal*, Boston Soc. of Civ. Engrs. (1920), Vol. VII, p. 47.

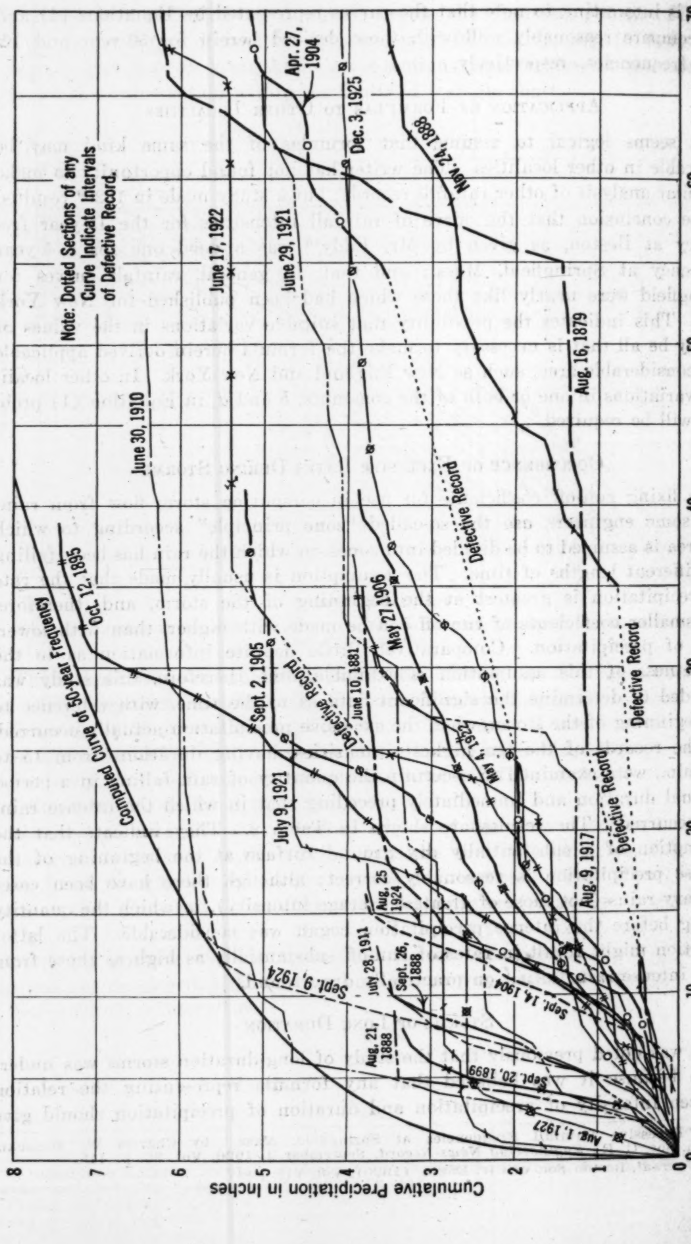


FIG. 2.—STORMS OF LONG DURATION, CHESTNUT HILL, BOSTON, MASS., 1879 TO 1928, INCLUSIVE.

TABLE 2.—Quantities of Rainfall in Periods of Equal Duration Immediately Preceding That in Which the Intense Rainfall Occurred.

Order of magnitude.	Date.	Intensity, in inches per hour.	Total quantity before maximum period from beginning of storm, in inches.	Quantity in equal period preceding the maximum period, in inches.	Quantity during period of maximum rate, in inches.	Average intensity during equal period preceding maximum period, in inches per hour.
15-Minute Intensities						
1	Aug. 22, 1899	5.67	0.26	0.26	1.42	1.04
2	July 9, 1921	4.16	0.30	0.24	1.04	0.96
3	July 20, 1880	4.14	0.38	0.29	1.035	1.16
4	July 21. 1894	3.50	0.20	0.20	0.88	0.80
5	Aug. 25, 1901	3.44	0.47	0.18	0.86	0.72
6	July 18, 1884	3.42	0.35	0.02	0.86	0.08
7	Aug. 6, 1893	3.36	0.00	0.00	0.84	0.00
8	Sept. 25, 1911	3.04	0.27	0.27	0.76	1.08
9	July 10, 1921	3.00	1.02	0.18	0.75	0.72
10	Aug. 12, 1892	2.78	0.01	0.01	0.69	0.04
30-Minute Intensities						
1	Aug. 22, 1899	3.51	0.00	0.00	1.76	0.00
2	July 9, 1921	3.18	0.30	0.30	1.59	0.60
3	July 20, 1880	2.71	0.08	0.07	1.36	0.14
4	Sept. 25, 1911	2.63	0.14	0.14	1.32	0.28
5	Aug. 12, 1892	2.50	0.01	0.01	1.25	0.02
6	Aug. 25, 1901	2.31	0.33	0.12	1.155	0.24
7	July 21, 1894	2.22	0.00	0.00	1.11	0.00
8	Aug. 10, 1917	2.05	0.65	0.25	1.03	0.50
9	July 10, 1921	1.99	0.84	0.06	0.99	0.12
10	Aug. 21 1888	1.96	1.75	0.55	0.98	1.10
45-Minute Intensities						
1	July 9, 1921	2.78	0.12	0.12	2.09	0.16
2	Aug. 22, 1899	2.49	0.00	0.00	1.87	0.00
3	Aug. 25, 1901	2.03	0.32	0.21	1.52	0.28
4	July 20, 1880	1.98	0.01	0.01	1.45	0.01
5	Sept. 25, 1911	1.93	0.01	0.01	1.45	0.01
6	Aug. 12, 1892	1.84	0.01	0.01	1.38	0.01
7	Aug. 21, 1888	1.72	1.75	0.61	1.29	0.81
8	June 30, 1915	1.62	0.63	0.34	1.22	0.45
9	Sept. 7, 1891	1.61	0.42	0.10	1.21	0.13
10	Aug. 10, 1917	1.57	0.50	0.26	1.18	0.35
60-Minute Intensities						
1	July 9, 1921	2.20	0.06	0.06	2.20	0.06
2	Aug. 22, 1929	2.04	0.17	0.17	2.04	0.17
3	Aug. 25, 1901	1.80	0.24	0.02	1.80	0.02
4	Aug. 21, 1888	1.58	1.55	0.83	1.58	0.83
5	Aug. 12, 1892	1.57	0.01	0.01	1.57	0.01
6	June 30, 1915	1.57	0.63	0.40	1.57	0.40
7	July 20, 1880	1.50	0.01	0.01	1.50	0.01
8	Sept. 25, 1911	1.45	0.01	0.01	1.45	0.01
9	Sept. 7, 1891	1.39	0.39	0.13	1.39	0.13
10	June 19, 1884	1.33	0.09	0.09	1.33	0.09

TABLE 2.—(*Continued.*)

Order of magnitude.	Date.	Intensity, in inches per hour.	Total quantity before maximum period from beginning of storm, in inches.	Quantity in equal period preceding the maximum period, in inches.	Quantity during period of maximum rate, in inches.	Average intensity during equal period preceding maximum period, in inches per hour.
120-MINUTE INTENSITIES						
1	June 30, 1915	1.28	0.37	0.37	2.56	0.19
2	Aug. 21, 1888	1.22	0.93	0.85	2.44	0.43
3	July 9, 1921	1.18	0.00	0.00	2.36	0.00
4	Aug. 22, 1899	1.17	0.00	0.00	2.33	0.00
5	Sept. 20, 1899	1.07	0.63	0.55	2.14	0.28
6	Aug. 25, 1901	0.94	0.21	0.11	1.88	0.06
7	July 20, 1880	0.89	0.01	0.01	1.78	0.01
8	Aug. 10, 1917	0.88	0.34	0.15	1.76	0.08
9	July 9, 1921	0.86	0.24	0.01	1.72	0.01
10	Aug. 12, 1892	0.85	0.00	0.00	1.70	0.00
180-MINUTE INTENSITIES						
1	June 30, 1915	1.12	0.63	0.63	3.37	0.21
2	Aug. 21, 1888	1.00	0.36	0.36	3.01	0.12
3	Sept. 20, 1899	0.83	0.43	0.43	2.49	0.14
4	July 9, 1921	0.82	0.00	0.00	2.46	0.00
5	Aug. 22, 1899	0.78	0.00	0.00	2.33	0.00
6	Sept. 10, 1924	0.76	3.09	2.11	2.28	0.70
7	Aug. 1, 1927	0.68	trace	trace	2.04	0.00
8	Aug. 16, 1879	0.67	1.57	0.00	2.01	0.00
9	July 9, 1921	0.63	0.23	0.00	1.89	0.00
10	Aug. 10, 1917	0.63	0.25	0.25	1.89	0.08

as reasonable results for periods of 1 day, or more, as for 1 hour or 2 hours, and also because of the increasing use of rainfall records in estimating probable flood flows of streams. In order to visualize the way in which the rain falls in such storms, the records of cumulative amounts in the twenty-two storms of greatest significance which occurred at Chestnut Hill Reservoir in the 50-year period, 1879-1928, have been plotted in Fig. 2. The curves show the computed maxima to be equalled or exceeded at average intervals of 50 years, according to Equation (2).

ACKNOWLEDGMENTS

The writer is indebted to William E. Foss, Director and Chief Engineer, Water Division, Boston Metropolitan District, for permission to make use of the records contained in this paper, and to John W. Raymond, Assistant Engineer, Metcalf and Eddy, Boston, for assistance in the preparation of the paper.

DISCUSSION

W. W. Horner,[11] M. Am. Soc. C. E. (by letter).—Every designing engineer responsible for a combined or a storm sewer system would like to say:

"I am proportioning my sewers and drains to take the run-off corresponding to a 10-year frequency (or other frequency, as the case may be), because I have analyzed the conditions that will prevail when the less frequent and more intense precipitations occur.

"I find that rains corresponding to the 15-year curve will overcharge the sewers by a certain percentage; will cause a raising of the hydraulic grades which will back water into a certain percentage of the basements; and will create damage in the amount of so many dollars. I find that the still less frequent and more intense 20, 25, and 50-year types of rains will produce certain additional definite damages which I have evaluated.

"Summarizing all these damages, it appears that the average annual damage which will result from the occasional insufficiency of the sewers designed for the 10-year rain has a certain definite value, the capitalization of which is less than the cost of providing more capacious structures. Consequently, my choice of a 10-year frequency represents the exact economic solution of this particular problem."

It is highly improbable that any engineering designer has ever made any statement of this kind. Until now there has not been available a sufficiently good understanding of rainfall rates and frequencies to justify such study, nor has there been any indication that the character of land improvements within a drainage area can ever be so definitely known, and the use of the improvement so completely determined, that damages on the basis suggested could be arrived at with any reasonable approach to accuracy. Most engineers determine their choice on the knowledge of the temper of the citizenship and build sewers of a certain size, because they have come to believe that the use of smaller sizes would result in such frequent overcharge as to produce a mass of small damages and of great aggravations; and that the people are willing to pay for this particular type of sewer rather than to undergo the inconvenience and expense which the smaller and cheaper structures will involve.

It seems most unfortunate that a better economic analysis has not been possible. Although the extremes of detail set out in the hypothetical case noted are never expected to be available, it is desirable to approach that condition. Mr. Sherman's studies and results certainly offer at least a little advance toward the end desired.

There have been studies of rainfall frequencies of short storms in the past. Many engineers have delved into the matter crudely for office purposes. One very good preliminary study of this kind has been presented in the progress reports of the Committee on Rainfall and Run-Off, of the Municipal Engineers of the City of New York, but nothing heretofore has approached Mr. Sherman's work in the logic of handling, or in the careful detail of development.

If any difference of opinion were possible, it might lie in the placing of his curves with reference to points between 30 and 70 hours' duration.

---

[11] Chf. Engr., Sewers and Paving, St. Louis, Mo.

It looks as if his frequency curves are somewhat higher than the data justify for these longer storms, and this possibility is further borne out by the inspection of Fig. 2. If the storm of 1895 were considered as being possibly of the 100-year order, then his 50-year frequency envelope might well remain below the 7-in. line.

Considering the entire economic phase again, it is possible that the writer is over pessimistic as to the ultimate possibility of definite detailed analyses. While these are not likely to be used for newly developing areas, it is not impossible that matters involving relief sewers or flood channels for well developed districts of relatively unchanging use will occasionally be found to be susceptible to such treatment. When and if such occasions do arise, the author's data will be of inestimable value.

C. S. JARVIS,[12] M. AM. SOC. C. E. (by letter).—The author has set forth in summarized form some of the outstanding meteorological data with which a designer of waterways should be concerned. Only after a thorough analysis and concentration, comparable with the well-known process developed by the mining industry, will the greatest values of such records be made available. While discoveries or observations locate and define the immense volumes in which values may be found, the processes of reduction and separation bring forth the precious elements from the inert mass.

Some of the quantities displayed in Fig. 1 are re-arranged in Table 3.

TABLE 3.—MAXIMUM RAINFALL INTENSITIES AT BOSTON, MASSACHUSETTS.

(Expressed in inches per hour.)

Rainfall periods.	Frequencies, in Years.							
	1.	2.	5.	10.	16⅔.	25.	50.	100.
5 min.	2.8	3.3	4.3	5.3	5.8	6.7	8.1	10.0
15 min.	1.85	2.3	2.8	3.4	3.8	4.3	5.4	6.5
60 min.	0.83	1.0	1.3	1.6	1.75	2.0	2.4	2.9
3 hours	0.40	0.48	0.63	0.75	0.84	0.98	1.20	1.50
24 hours	0.10	0.13	0.16	0.18	0.20	0.24	0.27	0.34

This table shows that the maximum intensity to be expected during a century is about double that of a 10-year period, or four times that of yearly frequency. Conversely, a 10-year maximum intensity is to that of a 1-year frequency as a century maximum is to a 10-year maximum. Furthermore, a 50-year maximum is approximately double that of a 5-year maximum; and, similarly, for any other periods differing by a factor, 10. The logical assumption from the foregoing is that the 1 000-year maximum would be double the intensity listed in Table 3 recorded for a century, for each designated period, whether for minutes, hours, or for a day.

Examining the various columns of Table 2, it is apparent that the summations of maximum rates for the ten recorded storms during 15, 30, 45, 60, 120, and 180-min. intervals, respectively, are nearly 5.5, 8, 9, 9.5, 10, and 6.5 times greater than for the equal periods immediately preceding the intense rainfall. This is quite in accordance with the usual rainfall habits in temperate zones. The first few drops serve as a warning even though the sky may be bright or only slightly cloudy overhead. As the storm cloud advances the curtain of rain often inclines at a considerable angle, or whips about similarly to window draperies. Later, the brief intermittent squalls are displaced either by a steady downpour, or by a period of calm succeeding the storm, depending on the location of the atmospheric disturbance, its duration, and the path described.

---

[12] Prin. Hydr. Engr., U. S. Engr. Office, War Dept., Washington, D. C.

It is re-assuring to find some semblance of consistency even among habits proverbially regarded as fitful and uncertain, such as those displayed by rainfall. The author has performed a lasting service by his presentation of the subject.

MILTON F. WAGNITZ,[13] M. AM. SOC. C. E., and LEWIS C. WILCOXEN,[14] ASSOC. M. AM. SOC. C. E. (by letter).—This paper is of particular importance. It establishes a basis by which the rainfall characteristics of different parts of the United States may be compared, and paves the way for determining the general laws of rainfall.

The writers' discussion is limited to the analysis of the rainfall data of Detroit, Mich. Before submitting the results for comparative purposes, an explanation of these data will be made. The statistical method used will also be outlined because it is believed that, for comparative purposes, the more exact results obtained are decidedly worth the labor. The rainfall data used are those of the U. S. Weather Bureau Station, situated on the Majestic Building, in Detroit. The records cover the 30-year period from 1896 to 1925, inclusive.

Records of excessive storms were tabulated by standard methods, that is, the period rates of each storm were tabulated, beginning with the period of maximum intensity. The periods taken were 5, 10, 15, 20, 30, 45, 60, 80, 100, 120, 150, 180, 210, 240, 270, 300, and 360 min. Intensities which were lower than those of succeeding periods were tabulated on the principle of extended duration, that is, the period of low intensity was credited with the rate of the higher succeeding period. The 1-year storm was taken as that hypothetical storm, the period intensities of which were equalled or exceeded thirty times. The 2-year storm is the one the period intensities of which were equalled or exceeded fifteen times, etc. The hypothetical frequency storms determined on this basis are shown on Fig. 3. It will be seen that the higher frequencies become progressively irregular. This, of course, should be expected since data by which they are determined, decrease in proportion to the frequency.

In fitting a system of curves to these data, it is believed that the importance of each frequency curve should be weighted, according to the number of storms by which it was determined. The determination of the weighted average frequency curve was, therefore, the first step in the analysis.

In the logarithmic form, Equation (1) (which is the general curve to be fitted) becomes $\log i = \log K + d \log (t + b)$. A value of 6 was first assumed

---

[13] Engr. of Public Structures, City Engr.'s Office, Detroit, Mich.

[14] Asst. Civ. Engr., City Engr.'s Office, Detroit, Mich.

for b, and K and d were determined by least squares. The normal equations for this solution are:

$$\log i = n \log K + d\,\Sigma\,[\log (t + 6)] \ldots\ldots\ldots\ldots\ldots (6)$$

and,

$$\Sigma\,[\log i \times \log (t + 6)] = \Sigma\,[\log (t + 6)] \log K + d\,\Sigma\,[\log (t + 6)]^2 \ldots (7)$$

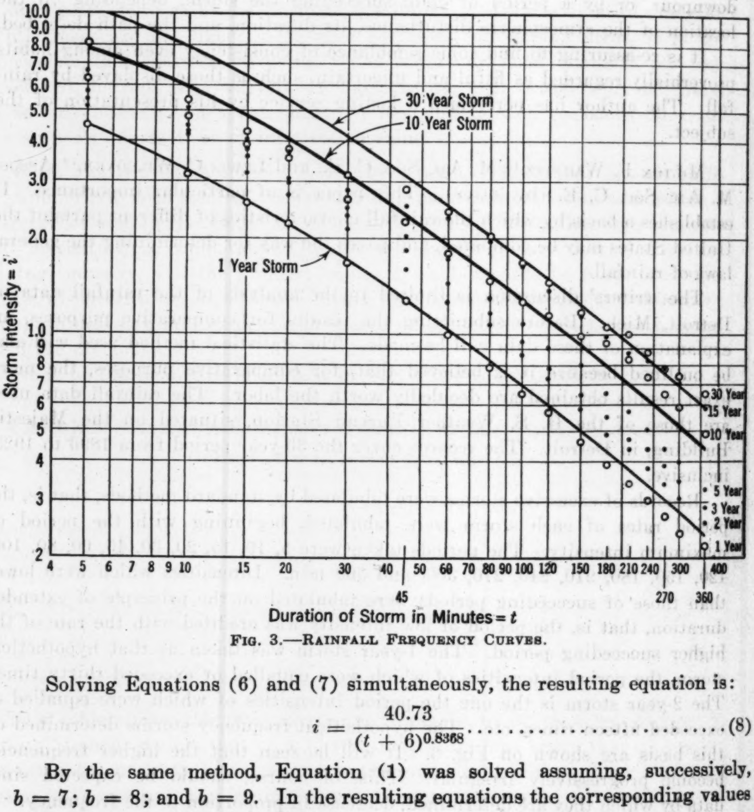


FIG. 3.—RAINFALL FREQUENCY CURVES.

Solving Equations (6) and (7) simultaneously, the resulting equation is:

$$i = \frac{40.73}{(t + 6)^{0.8368}} \ldots\ldots\ldots\ldots\ldots (8)$$

By the same method, Equation (1) was solved assuming, successively, $b = 7$; $b = 8$; and $b = 9$. In the resulting equations the corresponding values of K and d are as listed, in Table 4.

The relative worth of these equations is determined by finding the squares of their respective standard errors $\left(\text{standard error} = \sqrt{\dfrac{\Sigma \text{ errors}^2}{n}}\right)$. These were next determined (see Table 4). The squares of these standard errors are plotted in Fig. 4. It will be noted that they are a minimum between $b = 8$ and $b = 9$, and very close to $b = 8$. Based on the 30-year rainfall record, the equation of the most probable rainfall curve for Detroit is: $i = \dfrac{K}{(t + 8)^{0.855}}$.

TABLE 4.—CONSTANTS FOR SUBSTITUTION IN EQUATION (1).

b.	K.	d.	Standard error.
6	40.73	0.8368	0.1185
7	43.62	0.8482	0.0720
8	45.80	0.8554	0.0292
9	52.75	0.8822	0.0415

The constants for the different frequency curves were next determined by the formula:

$$K = \frac{\Sigma\,[i\,(t+8)^{0.855}]}{n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

The results are noted in Column (2) of Table 5.

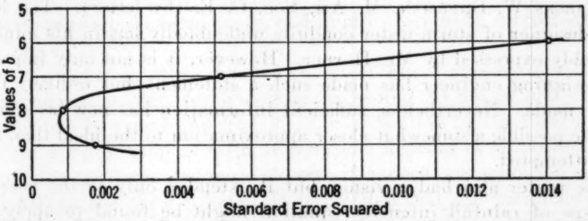


FIG. 4.—STANDARD ERROR, SQUARED, OF RAINFALL CURVE WITH VARIOUS VALUES OF b.

Fitting the curve, $K = a\,F^b$, to these data by least squares gives:

$$K = 37.6\,F^{0.263} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

TABLE 5.—FREQUENCY CONSTANTS FOR DETROIT RAINFALL CURVES.

Frequency, in years.	VALUES OF K (= $37.6\,F^{0.263}$).		Percentage error of actual K.
	Actual.	Computed.	
(1)	(2)	(3)	(4)
1....................	35.9	37.6	4.5
2....................	45.5	45.1	0.9
3....................	51.9	50.2	3.2
5....................	56.8	57.4	0.9
10....................	70.3	69.0	1.9
15....................	80.9	76.7	5.1
30....................	86.1	92.1	6.5
Average............	....	....	3.2

The various K values for the different frequency curves have been computed and are noted in Column (3) of Table 5, for comparison with the original data. To summarize the results of this analysis, it would appear:

1.—That the use of the least squares method for analysis of the data is desirable.

2.—That the Detroit rainfall curve has a different rate of change from that at Boston, Mass., and, hence, this may be expected to be different in different localities.

3.—That the relations of the intensities of the different frequency storms are almost exactly the same for Boston and Detroit, and, hence, that this may be a universal law.

4.—That because this frequency relation may be universal it seems likely that, if rainfall data were available covering a more extended period, the empirical rainfall curves of Detroit would more closely fit the curves herewith derived.

5.—That the data from other districts should be analyzed in order to verify Conclusions 1 to 4.

CHARLES W. SHERMAN, M. AM. SOC. C. E. (by letter).—The ideal which every designer of storm-water conduits undoubtedly has in his mind, has been admirably expressed by Mr. Horner. However, it is not only improbable that any designing engineer has made such a statement, but unlikely that it ever can be made. Nevertheless, sufficient information has now been accumulated to make possible a somewhat closer approximation to the ideal than has usually been attempted.

The writer also had a vision, but it extended only to the possibility that one type of rainfall intensity equation might be found to apply to a large section of the country—perhaps to the portion east of the Rocky Mountains—and that the frequency relation might be the same for all localities. If this were so it would be necessary only to determine the value of K_1 (in which, K corresponds to a frequency of one year), and all the necessary figures relating to rainfall intensity would be available. Moreover, analyses of precipitation records at a considerable number of stations would then make it possible to prepare a map showing lines for equal values of K_1, and interpolations between these lines would be sufficient, in most cases, to find a proper rainfall-intensity coefficient for almost any locality.

This vision may have been a mirage, and although the writer is disappointed, he is not surprised to learn that Messrs. Wagnitz and Wilcoxen do not find the same form of equation applicable at Detroit as that derived for Boston. That the frequency relation should be approximately the same in both cases, appears worthy of special notice. To be sure, the analysis of Detroit figures did not include any durations beyond 360 min., and it is possible that somewhat different results would have been obtained if longer storms had been taken into account.

The use of the method of least squares in computing the constants for the formulas, by Messrs. Wagnitz and Wilcoxen, appears to be advantageous, particularly in determining the best possible form of curve to fit a particular set of records. However, it appears from an examination of Fig. 3 that the form of curve resulting from these computations does not conform very well with the observed values of the first, second, and third magnitudes, and even the divergence from the points of thirtieth magnitude (1-year storm) is considerable. This may mean that for Detroit conditions the form of equation for each frequency is different from that for every other frequency. Perhaps, however, if the data were available for storms of longer duration, the apparent discrepancy between curves and plotted points would be less marked. It may be noted that if Fig. 1 had extended only to 3-hour storms, the divergence between the curves and the observations would have been more noticeable than is actually the case.

In view of the many items in storm-water design, which have to be fixed by the exercise of judgment, it is not impossible that a formula for intensity-duration relation of the form suggested in the paper may be close enough for general use; that the denominator should be $(t + 7)$, or $(t + 8)$, with an exponent of $\frac{3}{4}$, $\frac{4}{5}$, or $\frac{5}{6}$; and that the frequency relation may be taken as a function of $F^{\frac{1}{4}}$. Such figures, like the $\frac{3}{2}$-exponent in the usual weir formula, would be easy to remember and to apply, and might prove to be correct enough for ordinary use.

Mr. Horner is right in noting that the curves of Fig. 1 appear to be higher than the observations justify, for durations exceeding 30 hours, and this is particularly obvious in examining Fig. 2. A modification of the form of the curve, requiring a more complex form of equation, appears to be necessary if the points are to be approximated more closely; if, indeed, any equation can be found which will fit the observations. It is possible that the parallel curves obtained from the Boston observations represent a fortuitous circumstance, and that records at other places will not show similar results. Until the records of a considerable number of stations have been analyzed, it is possible only to speculate on this point.

Mr. Jarvis' comparison of the relations between intensities and frequencies is interesting, and is in such a form as to be easily remembered and applied. Since the Detroit data furnish some degree of confirmation of the figures obtained for Boston, these relations may be considered more likely to be of general application than any of the other figures presented. Whether the "semblance of consistency" in the intensity-duration relation is really reassuring, or just interesting and suggestive, can only be known when other records have been studied in a similar manner.

The writer agrees with Merrill M. Bernard, M. Am. Soc. C. E.,[15] that the intensity-duration formulas of the types, $i = \frac{K}{t + b}$ and $i = \frac{K}{t^d}$, are not equally applicable to both long and short periods of duration; but he believes that the type, $i = \frac{K}{(t + b)^d}$, is equally applicable to both long and short durations. Obviously, when t is large and b is less than 10, the difference between t^d and $(t + b)^d$ is insignificant.

There is considerable probability of error in using the Morgan and Meyer data in the way Mr. Bernard has done in the paper cited, first, because few,

---

[15] "Formulas for Rainfall Intensities of Long Duration," *Proceedings*, Am. Soc. C. E., October, 1930, Papers and Discussions, p. 1835.

if any, "1-day", "2-day", or "3-day" storms lasted exactly 24, 48, or 72 hours; and, second, because the estimation of frequencies by computations based upon the sum of the length of records at several stations is open to severe criticism. Obviously, the sum of 100 one-year records, all for the same year, at 100 stations, cannot be equivalent to a record of a century at a single station. However, taking the figures given in Mr. Bernard's Table 3[16] for Quadrangle 15-*E*, and leaving out of consideration the slight differences in exponent of the denominator, it would appear that the frequency relation would be a function of $F^{0.22}$. Possibly this may be taken as further confirmation of the idea that the frequency relation may generally approximate a function of $F^{\frac{1}{4}}$.

---

[16] *Proceedings*, Am. Soc. C. E., October, 1930, Papers and Discussions, p. 1842.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No 1781

THE TRAINING WALL ACROSS THE LIAO BAR IN MANCHURIA[1]

By P. N. Fawcett,[2] M. Am. Soc. C. E.

With Discussion by Messrs. R. D. Goodrich, Herbert Chatley, H. C. Ripley, B. Okazaki, and P. N. Fawcett.

Synopsis

This paper describes in detail the problems in harbor work encountered at the Port of Niuchuang in Southern Manchuria. The discharge of the Liao River has been such as to form a bar at its mouth, that continually hampers free navigation. The paper presents a description of the movement of this bar before the construction of the single training wall as well as during its construction. It also contains supplementary data concerning tides, winds, rainfall, velocities of river flow, and other information important to a study of this kind.

Location

The Liao River is the most important waterway in Manchuria. Its length is 600 miles (of which 350 are navigable for junk traffic) and its drainage area is 80 000 sq. miles. It empties into the northern extremity of the shallow Gulf of Liaotung and the Gulf of Pechihli. The Port of Niuchuang (Newchwang), for which the improvements described in this paper were made, is shown in Fig. 1, at the northern end of the Gulf of Liaotung.

Tidal Data

Fig. 2 shows the tidal curves for this river. Spring tides range between 10 and 13 ft. and neap tides between 7 and 8 ft.; the diurnal inequality during the spring tides is sometimes as much as 4 ft., the variation being confined to the high waters, with practically none in the low waters. Owing to the peculiar formation of the entrance to the Gulf of Pechihli the tides are greatly influenced by winds. A northerly wind drives the water out of the Gulf, thus reducing the depths along the coasts, while a southerly wind forces the water into it, between Korea and the Shantung Peninsula, thereby augmenting the depth all over this shoal Gulf. Fig. 3 contains curves of simultaneous tidal lines and of high-water and low-water planes corresponding to Fig. 2. The readings are for stations along the river from about 6 miles below the Signal Station to a station at Uu-tai-tzu, about 65 miles up stream from Swan Island (see Fig. 4).

---

[1] Published in May, 1930, *Proceedings*.

[2] Engr. in Chf., Lower Liao River Conservancy, Niuchuang, Manchuria, North China; Care, F. Hewett, The Windmill, Hythe, near Southampton, England.

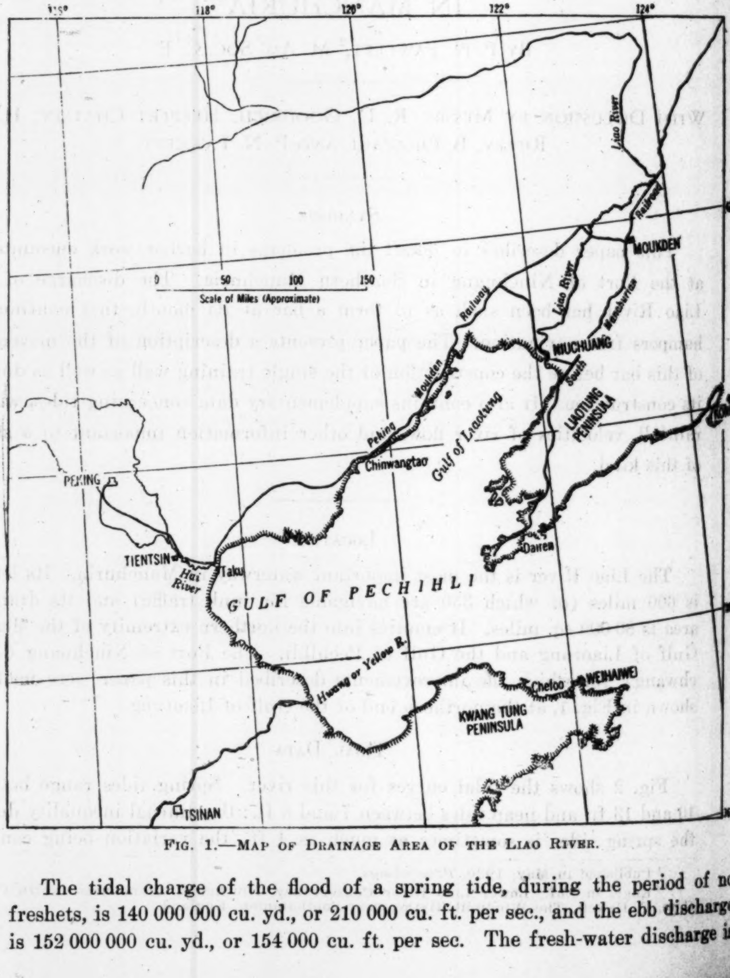

118°
120°
122°
124°
Liao River
Railroad
MOUKDEN
0 50 100 150
Scale of Miles (Approximate)
Railway
Liao River
Manchurian
Moukden
NIUCHUANG
South
LIAOTUNG PENINSULA
KOREA
Peking
Gulf of Liaotung
Chinwangtao
PEKING
Dairen
TIENTSIN
Hai River
Taku
GULF OF PECHIHLI
Hwang or Yellow R.
Cheloo
WEIHAIWEI
KWANG TUNG PENINSULA
TSINAN

FIG. 1.—MAP OF DRAINAGE AREA OF THE LIAO RIVER.

The tidal charge of the flood of a spring tide, during the period of no freshets, is 140 000 000 cu. yd., or 210 000 cu. ft. per sec., and the ebb discharge is 152 000 000 cu. yd., or 154 000 cu. ft. per sec. The fresh-water discharge is

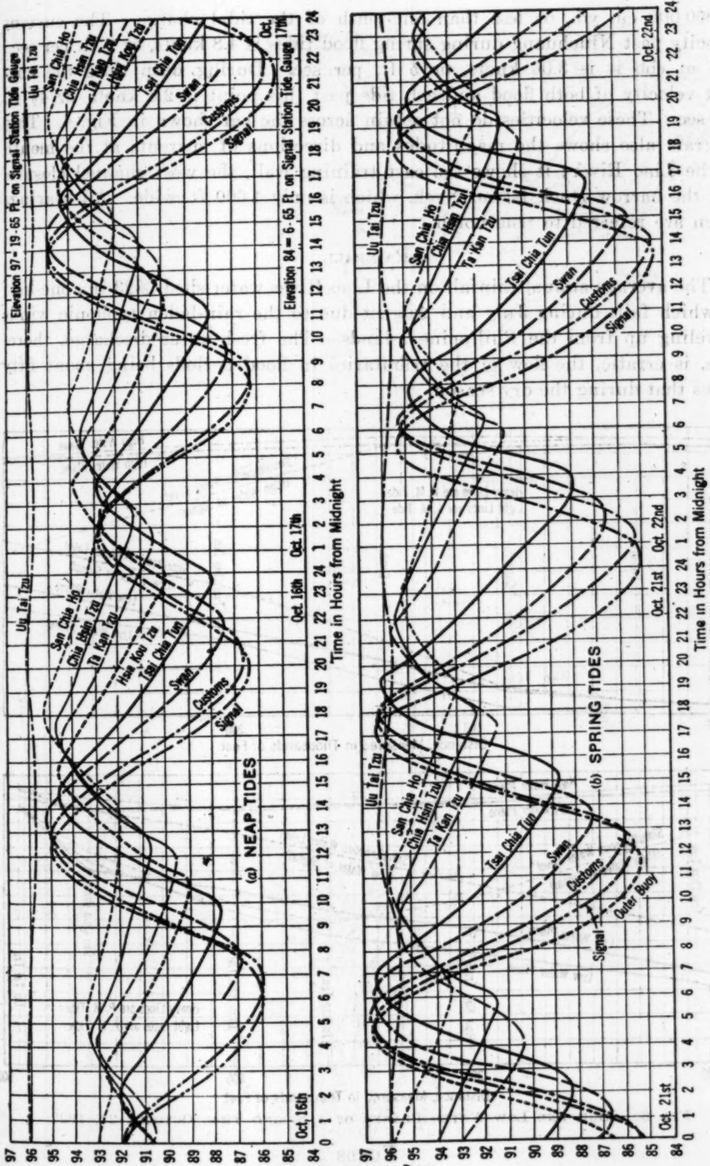


FIG. 2.—TIDE CURVES FOR THE LIAO RIVER.

12 000 000 cu. yd., or less than one-tenth of the tidal charge. The current velocity past Niuchuang during spring flood tides is 4.8 knots, or 8 ft. per sec., and at ebb it is 3.06 knots, or 5 ft. per sec. During neap tides the current velocity of both flood and ebb tide past this point is 2.6 knots, or 4½ ft. per sec. These velocities do not obtain across the bar shown in Fig. 4. This diagram also shows the magnitudes and directions of currents at the mouth of the Liao River; it shows the east training wall, the west channel closure, and the narrow Duck Island Neck, which is only 1 600 ft. wide. All bearings given are referred to true north.

RAINFALL

The average annual rainfall on the Liao River water-shed is 23 in., one-half of which falls during July and August due to the rain-laden typhonic winds traveling up from the Philippine Islands. The fresh-water discharge, therefore, is erratic, the flow of the tributaries in flood periods being about fifty times that during the dry season.

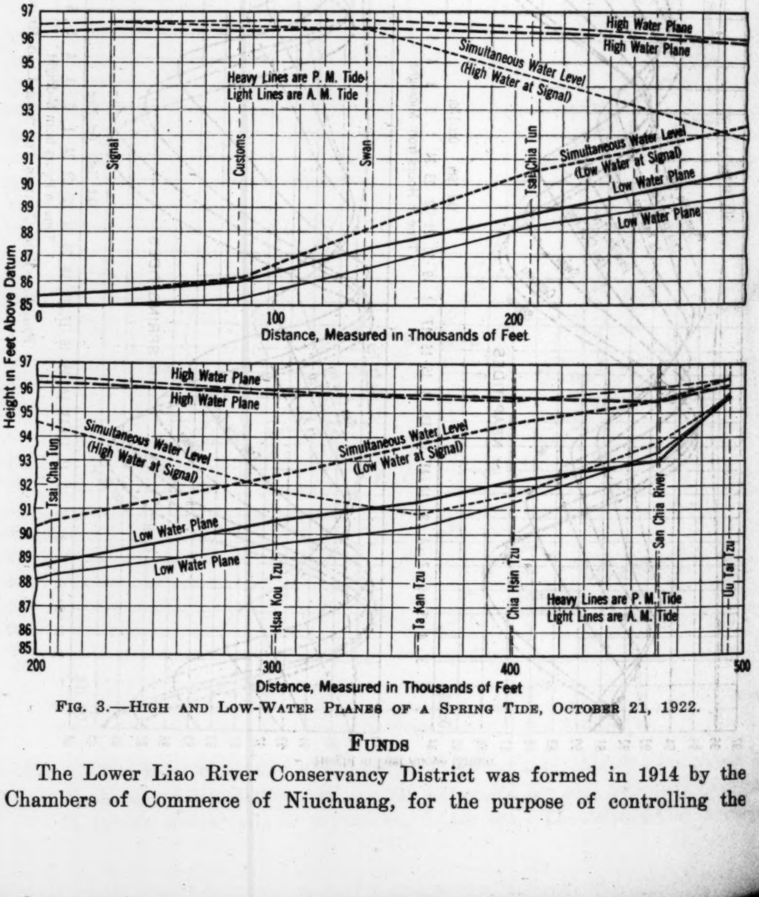


FIG. 3.—HIGH AND LOW-WATER PLANES OF A SPRING TIDE, OCTOBER 21, 1922.

FUNDS

The Lower Liao River Conservancy District was formed in 1914 by the Chambers of Commerce of Niuchuang, for the purpose of controlling the

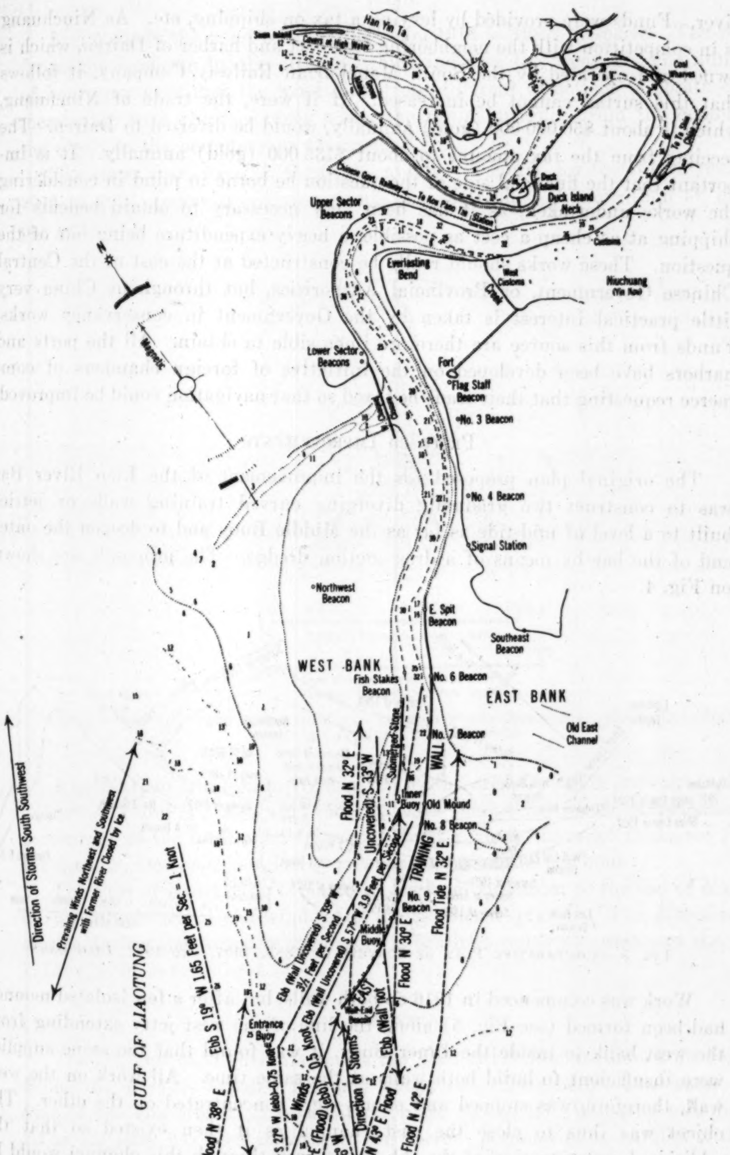

Han Yin Ta
Swan Island
Covers at High Water
Chinese Govt. Railway — To Kou Pang Tzu (Station)
Coal Wharves
To Tu Sha Creek
Duck Island
Duck Island Neck
Upper Sector Beacons
Customs
Everlasting Bend
West Customs
Creek
Niuchuang (Yin Kow)
Lower Sector Beacons
Fort
Flag Staff Beacon
No. 3 Beacon
No. 4 Beacon
Signal Station
Northwest Beacon
E. Spit Beacon
Southeast Beacon
WEST BANK
Fish Stakes Beacon
No. 6 Beacon
EAST BANK
Submerged Stone
No. 7 Beacon
Old East Channel
Flood N 32° E
Uncovered) S 33° W
TRAINING WALL
Inner Buoy
Old Mound
No. 8 Beacon
Flood Tide N 32° E
No. 9 Beacon
Middle Buoy
Ebb (Wall Uncovered) S 59° W 3½ Feet per Second
Ebb (Wall Uncovered) S 57° W 3.9 Feet per Second
Flood N 30° E
Direction of Storms South Southwest
Prevailing Winds Northeast and Southwest with Former River Closed by Ice
Ebb S 19° W 1.65 Feet per Sec. = 1 Knot
GULF OF LIAOTUNG
Entrance Buoy
EAST
Ebb (Wall not
Flood N 38° E
Prevailing Winds S 27° W (ebb)—0.75 Knot
N 38° E (Flood) (ebb) 0.5 Knot
Direction of Storms S 16° W
N 43° E Flood
Flood N 42° E
Lightship Niuchuang

FIG. 4.—DIRECTIONS AND SPEEDS OF CURRENTS IN THE VICINITY OF LIAO BAR.

river. Funds were provided by levying a tax on shipping, etc. As Niuchuang is in competition with the neighboring free port and harbor of Dairen, which is owned and operated by the South Manchurian Railway Company, it follows that this surtax cannot be increased. If it were, the trade of Niuchuang, which is about $50 000 000 (gold) annually, would be diverted to Dairen. The receipts from the tax amount to about $135 000 (gold) annually. It is important that the financial side of the question be borne in mind in considering the works undertaken, as it has been very necessary to obtain benefits for shipping at as cheap a cost as possible, a heavy expenditure being out of the question. These works should really be constructed at the cost of the Central Chinese Government, or Provincial Authorities, but throughout China very little practical interest is taken by the Government in conservancy works. Funds from this source are therefore impossible to obtain. All the ports and harbors have been developed on the initiative of foreign chambers of commerce requesting that their trade be taxed so that navigation could be improved.

PROPOSED IMPROVEMENTS

The original plan proposed for the improvement of the Liao River Bar was to construct two gradually diverging curved training walls or jetties built to a level of mid-tide as far as the Middle Buoy and to deepen the outer end of the bar by means of a drag-suction dredge. The proposals are shown on Fig. 4.

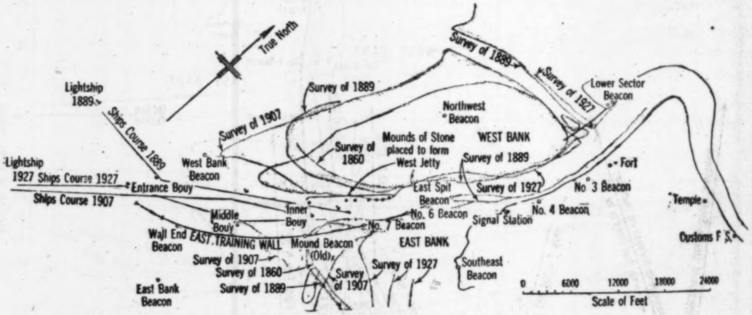

True North
Lightship 1889
Ships Course 1889
Lightship 1927 Ships Course 1927
Entrance Bouy
Ships Course 1907
Survey of 1889
Survey of 1907
West Bank Beacon
Survey of 1860
Mounds of Stone placed to form West Jetty
Survey of 1889
Survey of 1889
Survey of 1927
Lower Sector Beacon
Northwest Beacon
WEST BANK
Survey of 1889
Fort
East Spit Beacon
Survey of 1927
No 3 Beacon
Inner Bouy
Middle Bouy
No. 6 Beacon
No. 4 Beacon
Temple
Signal Station
No. 7 Beacon
Wall End Beacon
EAST TRAINING WALL
Mound Beacon (Old)
EAST BANK
Customs F S.
Survey of 1907
Survey of 1860
Survey of 1889
Survey of 1907
Survey of 1927
Southeast Beacon
East Bank Beacon
0 6000 12000 18000 24000
Scale of Feet

FIG. 5.—COMPARATIVE PLAN OF SURVEYS OF 1889, 1907, AND 1927, LIAO BAR.

Work was commenced in 1916 on both walls, but after a few isolated mounds had been formed (see Fig. 5) along the line of the west jetty extending from the west bank to inside the Inner Buoy, it was found that the stone supplies were insufficient to build both walls at the same time. All work on the west wall, therefore, was stopped and efforts were concentrated on the other. The object was thus to close the east channel as it then existed so that the additional concentration of the ebb that flowed through this channel would be transferred through the ships' course.

When this channel was closed by the east jetty, which was constructed as far as the Middle Buoy, the west jetty was then to be completed.

Wind

Fig. 6 shows the wind rose for the years 1916 to 1927, inclusive. The wind frequency is shown by the area, *B*, which includes the area, *A*. The number of hours per year that the wind blew from different directions is obtained by measuring, with the frequency scale, the radial distance from the center of the circle to the boundary of the area, *B*.

The inner rose, Area *A*, shows the number of hours, for the twelve years—1916 to 1927, inclusive—during which the wind blew at more than 56 miles per hour.

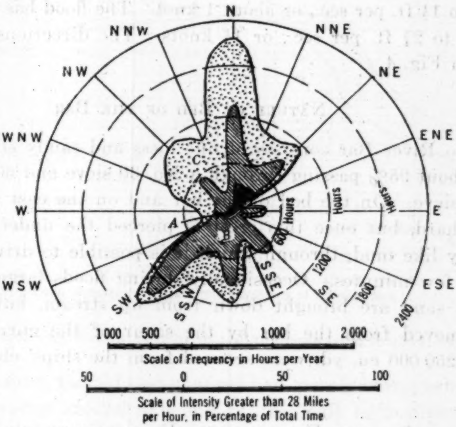


FIG. 6.—ANNUAL WIND FREQUENCY AND INTENSITY DIAGRAMS FOR YEARS 1916–1927, OBSERVED AT NIUCHUANG CUSTOMS.

A measure of wind intensity is given by the area, *C*, which does not include the areas, *A* and *B*. Measured radially, with the intensity scale, it indicates the percentage of the total wind frequency during which the wind measured from the area, *B*, blew with a velocity greater than 28 miles per hour.

The Port of Niuchuang is closed by ice from December to the end of March, during which period the wind is in the northeast quarter. The direction of storms during the period of open port is south to south 22° west and the prevailing wind is southwest.

Currents

The ebb and flood currents across the Liao River Bar are shown on Fig. 4. Since the jetty is exposed for only about two hours before low water, the first of the ebb crosses it in a direction, S. 30° to 35° W. (true meridian). After the jetty has been uncovered, the current becomes more parallel with it, that is, about S. 55° W.

On the seaward side of the entrance buoy and beyond the influence of the jetty, the ebb currents flow S. 16° to 33° W.; the more southerly direction being the ebb current flowing down and practically parallel with the west side of the west sand-bank. The flood tides flow in a direction from N. 25° to 35° E.

Velocities of Flow

During spring tides the velocity of the ebb current within the influence of the training wall is from 2½ to 3½ ft. per sec., or a mean of 3 ft. per sec. The velocity of flow at the bottom is 2 ft. per sec. On the seaward side of the Entrance Buoy and along the west side of the west sand-bank, the velocity is from 1 ft. to 1½ ft. per sec., or about 1 knot. The flood has a speed throughout of from 2¼ to 2¾ ft. per sec., or 1½ knots. The directions and velocities are as shown on Fig. 4.

Nature of Bed of the Bar

The Liao River Bar consists of fine loess and sandy silt brought down by the river, about 95% passing through a No. 30 sieve and 50% passing through a No. 175 sieve. On the bed of the bar and on the east and west banks the surface is hard, but once this crest is pierced the underlying material is a gummy clay like mud, through which it is possible to drive a water jet down 20 ft. in a few minutes. Occasionally during floods large quantities of pure fresh-water sand are brought down from up stream, but such deposits are quickly removed from the bar by the scour of the currents. In 1920, for example, 1 250 000 cu. yd. were scoured from the ships' channel in the course of 15 days.

Littoral Drift and the Formation of the Bar

There appears to be no littoral drift at the Liao Bar and comparative surveys (see Fig. 5) do not show a definite relation between storms and changes in the low-water contours or depths. Again, contrary to the generally expressed opinion that bars at the mouths of tidal rivers are formed of the materials thrown up from the ocean bed during storms, those at the entrances of the Liao, Hai, and Yellow Rivers, flowing into the Gulf of Pechihli, are all formed by alluvia brought down by the rivers themselves and deposited in the sea in a similar manner to deltaic formations at the mouths of non-tidal rivers. The bars of the Liao and Hai Rivers both shoal from 1 ft. to 2 ft. during the freshet season which occurs in August and September, the period of calm weather. As far as can be ascertained storms appear to have very little effect upon the Liao Bar.

Improvement of the Bar by the Construction of the East Jetty Only

Fig. 7 shows three longitudinal profiles across the bar and along ships' courses as they were in the years 1917, 1927, and 1928 (see Fig. 5). In 1917, one year after construction had begun on the east wall, the bar was 2 miles long. At that time the crossing channel was only 6 ft. deep and during the greater part of the year it was only 5 ft. deep at low water of ordinary spring tides.

When a second survey was made in 1922 the wall had been constructed to a point opposite the Middle Buoy. Originally, this was to have been the end of two training walls, which were to have been assisted by a dredge.

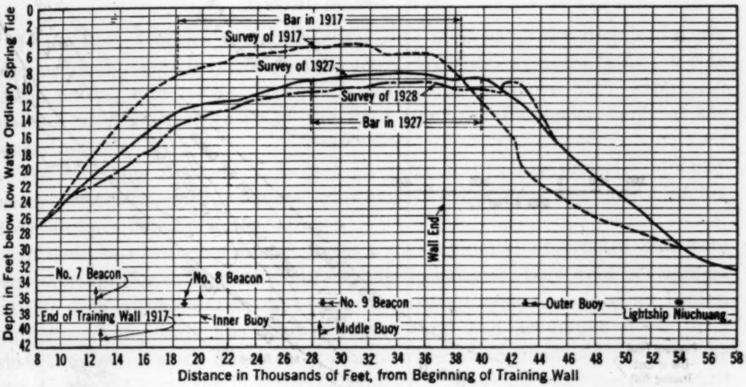


FIG. 7.—LONGITUDINAL SECTIONS ALONG LINE OF SHIPS' COURSES DURING 1917, 1927, AND 1928.

At that time, however, the old east channel had merely been pushed seaward and was still continuing around the end of the wall in a direction about S. 16° W. This is the same direction as the ebb currents in this vicinity and is in a mean direction of the prevailing winds and the storms.

The third survey made in 1924, after the wall had been extended still farther in a direction toward the Entrance Buoy, showed this easterly channel still persisting. During 1923, following a heavy freshet, the main body of the bar and ships' channel silted 2 ft. more than this easterly channel. Consequently, the easterly channel was made into the ships' course for a period of about one month until the silt and sand had been scoured off the bar. Then the old course was resumed and the construction of the wall was continued.

In 1926 the training wall was 6 miles long and a survey showed that the east channel was practically closed. An examination of these four surveys (1917, 1922, 1924, and 1926) showed that a depth of 9 ft. has always been obtained up to the existing end of the jetty and that by 1926 there was a navigable depth of 8 ft. across the bar. The positions of the bar in 1927 and 1928 are shown in Fig. 7. In 1928 the bar had a minimum navigable depth of 9 ft. and the wall was 37 750 ft. long.

The present (1928) plan is to terminate this wall in 14 ft. at low water opposite the Entrance Buoy (see Fig. 6). From Fig. 7 it will be noted that the bar has been advanced seaward into deeper water.

ADVANCE OF THE 9-FOOT CONTOUR

Fig. 8 shows the 9-ft. contour for a number of years from 1917 to 1928. The total advance is 22 500 ft. and has always followed the advance of the training wall. In fact, about 1 000 ft. up stream from the position of the contour for any year can be assumed to be the end of the wall for that year.

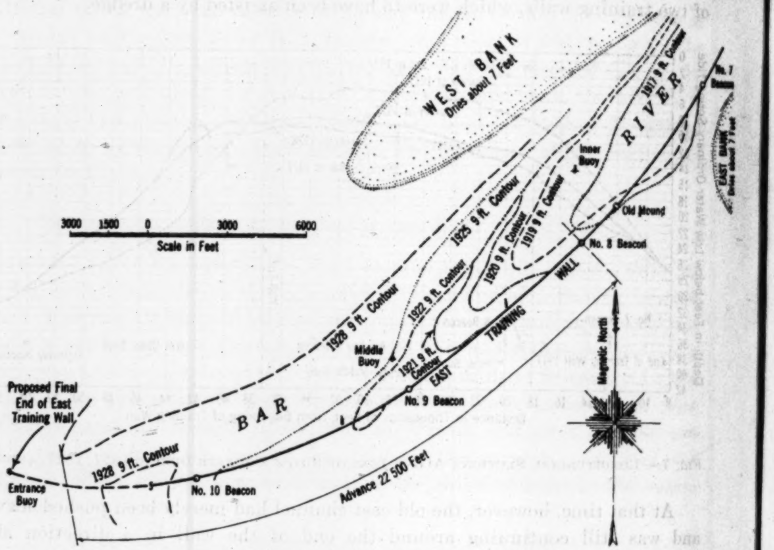

WEST BANK
Dries about 7 Feet
RIVER
1917 9 ft. Contour
No. 7 Beacon
EAST BANK
Dries about 7 Feet
Inner Buoy
Old Mound
No. 8 Beacon
1925 9 ft. Contour
1919 9 ft. Contour
1920 9 ft. Contour
1922 9 ft. Contour
1928 9 ft. Contour
3000 1500 0 3000 6000
Scale in Feet
Middle Buoy
1921 9 ft. Contour
EAST TRAINING WALL
No. 9 Beacon
Magnetic North
Proposed Final End of East Training Wall
BAR
1928 9 ft. Contour
Entrance Buoy
No. 10 Beacon
Advance 22 500 Feet

FIG. 8.—LIAO BAR: 9-FOOT CONTOUR FROM 1917 TO 1928.

The width from the wall to the west side of the contour (about 3 000 ft.) shows that the deepening has not been caused by over-fall during flood tide. The flood tide tops the wall after its first half hour of flow and passes over it very quietly and without any great disturbance. In another part of the river there is constructed a wall across what was known as the west channel. In this locality the early flood tide passes over with a head of about 1 ft. and causes a deepening on the river side due to over-fall. This deepening, however, is very narrow (about 100 ft.) and an excessive shoaling is formed in front of it.

Were the deepening across the bar caused by over-fall one would expect to find a corresponding shoaling in the main channel. The contrary is the case; the deepening that is taking place in the vicinity of the west bank extension is a disquietening feature because it is considered that should the channel deepen over too great a width, a shoaling will take place and necessitate the construction of a west wall. The deepening across the bar has undoubtedly been caused by the scour of the ebb tide excavating the bar and depositing it seaward into deep water.

QUANTITY SCOURED OFF THE BAR

The quantity scoured from the bar—irrespective of the annual freshet deposits—is equal to 22 000 000 cu. yd.

The annual freshet deposit is at times very great. In the freshet of 1923, surveys showed that a quantity of 1 250 000 cu. yd. of clean sand were removed in 15 days, or at the rate of 80 000 cu. yd. per day. Had such a deposit taken place prior to the construction of the wall, the bar would have shoaled for a year, or probably more, since there would have been no confined and trained ebb current to scour it away.

HEIGHT OF THE WALL

In the United States it would appear that high-water training jetties are advocated and almost invariably constructed. British engineers favor low-tide jetties, or half-tide jetties, because a jetty built to above high water will interfere with the free entry of the flood tide into the upper parts of the estuary or river.

The American practice of twin high-water jetties has proved eminently successful in numerous instances, of which two examples are the Columbia River entrance and Coos Bay, Oregon.

In these two instances single jetties were originally constructed and were very successful for some time. Unfortunately, however, they were not carried out into deep enough water; that is, to the neutral zone as defined by Italian engineers. The result was that after the sand had accumulated on their windward side so as to fill in the space provided, it commenced to enter the channel, thus causing the latter to deteriorate. A second jetty was then built extending out into deep water and the original jetty was also lengthened.

This work met with great success, but it seems unfortunate that the original single jetties were not extended first, so as to pass the sand drift around their outer ends, and so that the effects could be observed. By this means equally successful results might have been achieved without having recourse to the second jetty.

In the case of the Liao entrance the east and west banks form what are really two low-tide jetties which maintain a channel between their 20-ft. contours over a width of almost 2 000 ft. It would appear reasonable to expect two low-tide walls to act in a similar manner if extended seaward.

Irrespective of this point of view and because of ice drifting across the jetty during the winter months, it has been found that the wall across the Liao Bar must be either practically a low-tide one, or that it must be built completely above high water.

The present method is achieving immediate results while at the same time it permits the jetty to be carried above high water, should that be deemed advisable at any future date when the trade of the port becomes sufficient to justify the expense. Model tests would be invaluable in solving several problems of this nature.

The solution of the problem became a question of attempting to get the best results for a minimum expenditure. Since the whole body of the ebb tide travels in a south-southwesterly direction, or away from the west jetty (if that is ever built), and since it was seen that the low tide east jetty always deepened the bar to 9 ft., as its construction advanced, it was decided to continue extending the east jetty and to hold the construction of a west jetty in abeyance.

Before arriving at this decision, the question of costs was analyzed. To date (1928), the cost has been approximately $1 250 000 (gold). Maintenance charges are likely to equal about $60 000 (gold) per year, whereas the cost of a suitable drag suction dredge would be $650 000 (gold) and operating expenses $60 000 (gold) per year.

It would appear, therefore, to be more economical to abandon the intention of building the second jetty or heightening the present one and to rely on any further deepening to be obtained by dredging. This would save half the original first cost of the west jetty—or all the cost of heightening the east jetty—plus the additional interest and amortization charges. The latter would be sufficient to provide a new dredging unit every ten years.

COMPARATIVE SURVEYS

Comparative surveys previously mentioned, show that the 1-fathom depths have had an eastward advance of about 5 400 ft. between 1860 and 1927, or at the rate of about 80 ft. annually. The direction of the channel in 1860 and in 1889 was S. 53° W.; in 1907, it was southwest; and, in 1927, the channel trained by the east jetty was S. 67° W. Since 1907 the old east channel has been on the east side of the jetty. It is now filling rapidly with silt.

The writer has also studied the relative movement of the 2, 3, 4 and 5-fathom contours. They are of interest inasmuch as they show that the advance of the silt takes place southeast of the Entrance Buoy. The 3-fathom contour has moved seaward a distance of 8 000 ft. in 67 years, or at the rate of 120 ft. annually.

Furthermore, during the whole of this period, the contours north of the Entrance Buoy on the west side of the west sand-bank have maintained their position with very little change. This is believed to be due to the action of the flood and ebb tides passing straight up and down along these contours in a north and south direction, respectively, and thus carrying away any deposits that may be made from time to time in this locality.

LOCATION OF THE WALL

Hydraulic engineers are not in agreement as to the necessary direction and alignment of training walls across bars subject to tidal action. Some engineers recommend that the end of the wall be in line with and along the axis of the flood and ebb currents and directly facing into the direction of the prevailing winds and storms.[3]

Others favor inclining the wall away from the prevailing winds and heaviest seas and locating it so that these and the flood tide meet the wall slightly on its convex side.[4] This is the location that has been adopted in the case of the Liao Bar.

The survey made in 1927 shows clearly the steep slopes of the sea bed north of the entrance buoy (see Fig. 9, Curve *B D F*). The other surveys since 1860 show these slopes to have been maintained at the same location without practical alteration since that date. On the other hand, the sea bed slopes east of the jetty, along the axis of the east channel, and along the direction of the flood and ebb currents and the direction of prevailing winds and storms, are extremely flat, as shown by the longitudinal section, *C E F*, in Fig. 9.

---

[3] See Bibliography (Appendix), Items (3), p. 121; (6), pp. 303–304; (14), p. 354; (20), p. 194; (21), p. 99; (22); and (23), p. 207.

[4] See Bibliography (Appendix), Items (4), p. 485; (5), p. 536; (7), p. 6; (10), p. 10; (1); and (2).

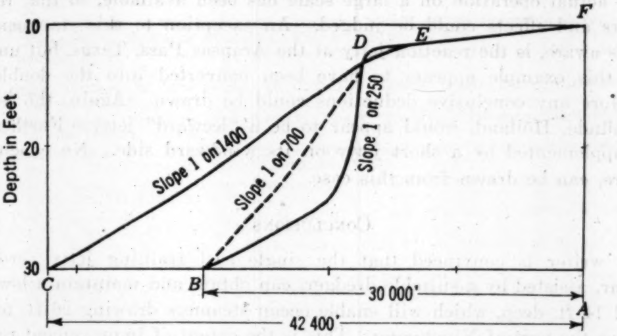


FIG. 9.—LONGITUDINAL SECTIONS NORTH OF THE ENTRANCE BUOY, LIAO BAR.

The question to be considered, therefore, was the correct direction to adopt. The old east channel course appears to be the natural direction of both the flood and ebb currents. This route would have provided a channel facing directly into the prevailing winds and greatest storms as advocated by some authorities; but it would also have led along a line of flat sea-bed slopes in a direction such that the greatest accretion and contour advance due to the deposits from the river would have been taking place.

The other alternative was to oppose the ebb currents by the training wall and attempt to guide the channel so as to terminate at or north of the Entrance Buoy in a locality of steep sea-bed slopes. As previously stated (see Fig. 9), the contours here have remained practically stationary for as far back as there are any records, that is, 1860. This procedure, on the other hand, would necessitate the flood tide, prevailing winds, and greatest storms being received on the convex side of the wall. Furthermore, the ebb currents on passing the end of the wall would have to make a more or less oblique bend to the southwest.

The pressure of the ice drift during the winter months and the abrasion against a wall that opposes the ebb currents in any way is very great and this no doubt increases the maintenance charges. Nevertheless, it was considered best to end the wall where the sea bed was steep because any other location would probably make it necessary to extend the wall continually to keep ahead of shoals formed by deposits brought down by the river. It was after consideration of these various factors that the present direction was adopted.

SINGLE OR DOUBLE TRAINING JETTIES

The question as to the advisability of constructing single or double training jetties at the entrances of tidal rivers for the purpose of providing a navigable channel across bars has been frequently discussed before the Society. Many eminent engineers have advocated a single jetty[5] and others, equally eminent, have advocated double jetties.[6]

The great difficulty has been that apparently no example of a single jetty in actual operation on a large scale has been available, so that relative behaviors and effects could be judged. An exception to this statement, the writer is aware, is the reaction jetty at the Aransas Pass, Texas, but unfortunately, this example appears to have been converted into the double-jetty type before any conclusive deductions could be drawn. Again, the jetty at Swinemünde, Holland, would appear to be a "leeward" jetty. Furthermore, it is supplemented by a short jetty on its windward side. No conclusions, therefore, can be drawn from this case.

CONCLUSIONS

The writer is convinced that the single east training jetty across the Liao Bar, assisted by a suitable dredger, can obtain and maintain a low-water channel 14 ft. deep, which will enable ocean steamers drawing 26 ft. to enter and leave the port of Niuchuang. This is the extent of improvement required in this particular case. As it is, vessels with a draft of as much as 22 ft. enter and leave the port, whereas previous to the construction of the jetty, 17½ ft. was the limiting draft.

APPENDIX

BIBLIOGRAPHY ON SINGLE AND DOUBLE TRAINING WALLS

The following is a bibliography of references favorable to single and to double training walls.

(1) **Brooks, William A.** On the Improvement of Tidal Navigations and Drainages. *Minutes of Proceedings*, Inst. C. E., Vol. XII (1852–53), pp. 1-23.

(2) **Wheeler, W. H.** Tidal Rivers. Longmans, Green and Co., Lond. and N. Y., 1893.

(3) **Symons, Thomas W.** Improving the Entrance to a Bar Harbor by a Single Jetty. *Transactions*, Am. Soc. C. E., Vol. XXXVI (1896), pp. 109 and 121. With discussion, p. 131.

(4) **Haupt, Lewis M.** The Reaction Breakwater as Applied to the Improvement of Ocean Bars. *Transactions*, Am. Soc. C. E., Vol. XLII (1899), p. 493. With discussion, pp. 503, 512, 516, 520; also, Vol. XLIII (1900), p. 102.

(5) **Harts, William W.** Description of Coos Bay, Oregon, and the Improvement of Its Entrance by the Government. *Transactions*, Am. Soc. C. E., Vol. XLVI (1901), p. 482. With discussion, pp. 526, 532.

(6) **Gillette, Cassius E.** Sea Coast Harbors in the United States. *Transactions*, Am. Soc. C. E., Vol. LIV, Pt. A (1905), pp. 317–321.

(7) **Vernon-Harcourt, L. F.** Construction and Maintenance of Seaports on Sandy Coasts. *Bulletin No. 61*, Inter. Nav. Congress (1908), p. 6.

(8) **Sanford, J. J.** Construction of Ports on Sandy Shores. *Bulletin No. 62*, Inter. Nav. Congress (1908), p. 8.

(9) **Gatto, D. Lo.** Construction of Ports on Sandy Shores. *Bulletin No. 63*, Inter. Nav. Congress (1908), p. 7.

(10) **Ivanina, A., and Aeckerle, W. A.** The Most Important Russian Ports on the Sandy Coasts of the Baltic. *Bulletin No. 65*, Inter. Nav. Congress (1908), p. 10.

(11) **Harts, William W.** Harbour Improvement on the Pacific Coast of the United States. *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXIII (1910–11, Pt. 1), pp. 237 and 239.

---

[5] See Bibliography (Appendix), Items (1); (2); (3), pp. 109, 121; (4), pp. 493, 512, 516, 520, 102; (12); (13); (14); and (23).

[6] *Loc. cit.*, Items (3), p. 131; (4), p. 503; (5) to (11); (15) to (20).

(12) **Halligan, G. H.** The Bar Harbors of New South Wales. *Minutes of Proceedings,* Inst. C. E., Vol. CLXXXIV (1910–11, Pt. II), pp. 128 and 205.

(13) **Wheeler, W. H.** Correspondence on Harbours in Australia. *Minutes of Proceedings,* Inst. C. E., Vol. CLXXXIV (1910–11, Pt. II), p. 277.

(14) **Tower, Morton L.** Notes on the Bar Harbors at the Entrances to Coos Bay, and Umpqua and Siuslaw Rivers, Oregon. *Transactions,* Am. Soc. C. E., Vol. LXXI (1911), p. 357.

(15) **Bagnall, Gerald.** The Improvement of the Mouth of the Columbia River. *Professional Memoirs,* U. S. Corps of Engrs., Vol. 8 (1916), p. 718.

(16) **Hickson, R. E.** Changes at Mouth of Columbia River, 1903 to 1921. *Military Engineer,* Vol. 14 (1922), p. 211.

(17) **Black, William M.** Formation of Channels Across Ocean Bars. *Military Engineer,* Vol. 14 (1922), p. 375.

(18) **Patton, R. S.** Problems Involved in Coast Erosion. *Military Engineer,* Vol. XVI. No. 90 (1924), p. 463.

(19) **Kuentz, Oscar O.** Cape Fear River Channel and Bar. *Military Engineer,* Vol. XVII, No. 91 (1925), p. 29. With discussion, p. 30.

(20) **Mayo, George.** The Improvements at Coos Bay, Oregon. *Military Engineer,* Vol. XVII, No. 93 (1925), p. 194.

(21) **Townsend, C. McD.** Problems at Mouth of the Mississippi. *Military Engineer,* Vol. XVIII, No. 98 (1926), p. 99.

(22) —— The Improvement of the Southwest Pass of the Mississippi River. *Bulletin No. 33,* Inter. Nav. Congress (1926).

(23) **Ripley, Henry C.** Relation of Depth to Curvature of Channels. *Transactions,* Am. Soc. C. E., Vol. 90 (1927), p. 235. With discussion, pp. 244–245.

DISCUSSION

R. D. Goodrich,[7] M. Am. Soc. C. E. (by letter).—A record of construction experience such as that presented in this paper, is always of value to the profession.

As shown in Fig. 1, there are four principal ports in the Gulfs of Liaotung and Pechihli. Dairen and Chinwangtao are located in natural indentations of the coast and are free from ice during the winter months. On the other hand, Niuchuang (Newchwang) and Tientsin have harbors in rivers that drain large areas and they are usually closed for several months in the winter. In many respects these two drainage systems are similar.

As the author states, the average fresh-water discharge of these rivers is not sufficient in itself to maintain proper channels for navigation by ocean steamers. The situation is made worse by the character of the floods, which carry enormous quantities of silt. Samples containing 10% by weight of silt are not uncommon. Commerce at Tientsin has been greatly handicapped in recent years by the silting of the channel from Tientsin to Taku, practically closing the Hai River to ocean steamers for months.

These conditions would naturally tend to the development of Dairen and Chinwangtao as the logical ports of North China. However, there are many other considerations having great weight in port development. Distribution to the interior from these two ports must of necessity be almost entirely by rail. On the other hand, Tientsin and Niuchuang have the decided advantage of inland waterways as well as railways for the collection and distribution of ocean freight. From Tientsin there are three waterways and three rail routes, north, south, and west, not all of which are shown on the outline map (Fig. 1). The greater cost of transportation by rail compared with that by Chinese river boats for many commodities, and the greater length of haul and consequent cost of freight from the winter ports to the points of distribution to the interior markets, largely offset the difficulty of maintaining river ports and harbors. There is, therefore, keen competition between these ocean terminals, influenced also by the large investments already made at each point. Not only must the present works be maintained, but improvements must be made in order to protect the business and investments in all these ports.

The most serious and difficult problem involved in the maintenance of these rivers and harbors is that caused by the formation of a bar outside the mouth of any of the larger streams of China. At the mouth of the Yellow River conditions are such as practically to prohibit the construction of a harbor for the interchange of freight between ocean steamers and river boats. The rate at which the coast line is being extended at this point is much greater than at the mouth of either the Hai River or the Liao River.

In 1925, the construction of training walls was also undertaken at the mouth of the Hai River. It is understood that this project contemplated the use of walls carried above high tide on both sides of the channel. The writer has always felt that a single curved jetty might have provided the desired benefits at much less cost to the interests involved. As noted by the author, all such local improvements must be financed largely by the collection of harbor dues, over and above the regular customs rates. It is evident, therefore, that development depends almost entirely upon the initiative and enterprise of the commercial interests, both Chinese and foreign, at each individual port. In earlier days dredging and other modern harbor works were instigated by the foreign shipping interests, but due to western influence Chinese capital is now also largely involved, the amount varying of course with the locality.

---

[7] Prof., Civ. Eng., Coll. of Eng., Univ. of Wyoming; Senior Hydr. Engr., U. S. Engr. Office, War Dept., Sacramento, Calif.

In view of all the difficult conditions connected with the maintenance and development of facilities for commerce in the Orient, the paper presents a very valuable record of experience and progress.

HERBERT CHATLEY,[8] ESQ. (by letter).—The problem of the Liao Bar typifies conditions in North China and provides an interesting commentary on the question of single or double training walls. The regulation of the bar of the Whangpoo River presents some analogies. Before regulation this bar had a lowest low-water depth on the crest of 15 ft. Between 1907 and 1909 a training wall 5 500 ft. long was built on the left bank above half-tide level with a radius of about 7 000 ft., extending to the 36-ft. lowest low-water contour of the Yangtze River, of which the Whangpoo is a tributary. This wall reduced the natural "flare" of the mouth by about 40% and its concavity concentrated both flood and ebb, with the result that by 1912 the lowest low-water depth on the crest was increased to 21 ft., at which it remained stationary for three years. The lowest low-water sectional area of the stream above the training wall is about 42 500 sq. ft. The tidal influx varies from 400 000 000 to 5 000 000 000 cu. ft. and the efflux from 1 500 000 000 to 4 400 000 000 cu. ft. The non-tidal discharge averaged 12 600 cu. ft. per sec.

The material of the bar consists of Yangtze mud, about 90% passing a 200-mesh screen. There is about 10% colloid which makes the material fairly cohesive. In shallow areas coarser particles may be segregated, forming patches of fine, fairly clean sand, but this is not found in the channel. The water is charged with silt which is brought in by the flood tide from the Yangtze in quantities sometimes greater than 500 parts by weight per million.

In 1914 a second training wall, 7 000 ft. long, to half-tide level was built on the right shore parallel to, and 2 500 ft. from, the other and, from that date to 1928, the lowest low-water depth on the crest of the bar increased to 30½ ft. Since then no appreciable change has occurred. This second wall reduced the low-water flare at the mouth by a further large amount, leaving only a taper of 1 in 100 between the walls. Parts of the areas between the walls and the old shore have since been reclaimed without appreciable effect on the bar.

While the conditions are very different from those of the Liao, the Whangpoo results show that the scour induced by a single concave wall can be usefully supplemented by the concentration of stream flow caused by a second wall.

---

[8] Engr.-in-Chf., Whangpoo Conservancy Board, Shanghai, China.

H. C. Ripley,[9] M. Am. Soc. C. E. (by letter).—This paper is a record, not of the method of improving the entrance to a harbor in the best and most economical way, but rather of securing the greatest channel depth at the entrance with the limited funds available. With this in view, the author seems to have met with considerable success.

The location and shape of the training wall are eminently suited for the purpose. The movement of material composing the bar at different stages of the tide is as follows: During flood the current crosses the wall at an angle, from the convex to the concave side, and by its overfall produces a deepening of the channel along the concave side and causes accretion on the convex side. During ebb exactly the opposite effects are produced until the tide has fallen sufficiently to uncover the top of the training wall. Then the wall becomes a real curved jetty and produces an effect as such. For the remaining two hours of the ebb, erosion takes place in the channel and the material thus excavated is carried across the channel and deposited on the convex side. Subsequently, a part of this material is washed back into the channel by the next returning ebb current, and the process is repeated by every recurring flood and ebb tide. However, the results have shown that the erosion in the channel is greater than the accretion; hence the beneficial effect of the training wall. The unfortunate feature of these effects is shown in the advance of the bar seaward, which will necessitate the continual extension of the training wall to keep up with the advance of the bar.

The author claims that no deepening of the channel is due to overfall during flood tide. It is believed, however, that the builders of dams who have to provide against the undermining action at the base of weirs and others familiar with the results of overfall will be loath to accept the conclusions of the author in this regard.

If, on the other hand, the plan had been designed to make the best improvement regardless of any limit as to expenditure—except that necessary to carry out the plan—it would have been for a curved jetty instead of the curved training wall and the work would proceed as follows: A foundation would be built 3 ft. in thickness and of sufficient width to prevent settlement of that portion upon which the superstructure would be located for the full length of the proposed jetty from the shore to a point a short distance beyond the crest of the bar. Commencing at the outer end, the superstructure would be built toward the shore to a height of high water of the greatest spring tide. In this way the advance of the bar would be prevented and no dredging would be required to develop the maximum channel that the curvature would permit.[10]

The cross-profile of the channel which a curved jetty will develop can be computed from the following formula:

$$Y = P\left(1 - \frac{X^2}{W^2}\right) + P\,\frac{26.28}{R}\left(1 - \frac{X^2}{W^2}\right)X \ldots\ldots\ldots\ldots (1)$$

in which, P is the mean depth of the channel multiplied by 1.65; W is the half width of the channel; R is the radius of curvature of the jetty; and X and Y are the co-ordinates of the cross-profile, the origin being at the center of the channel at the surface of the water.

---

[9] Cons. Civ. Engr. (*Retired*), Brooklyn, N. Y. Mr. Ripley died January 9, 1931.

[10] See *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 1040, also Vol. 90 (1927), p. 207.

The value of P must be determined from the cross-section of the channel at the mouth when the river is at a high flood stage and the spring tide is at its maximum.

The navigability of the channel for the deeper-draft vessels entering the port may be increased to a limited extent by diminishing the width of the channel at the river, thus increasing the value of P and reducing the value of W.

A further enlargement of the channel if desired can be effected by dredging. This enlargement will be maintained by the jetty without additional cost for maintenance.

B. Okazaki,[11] M. Am. Soc. C. E. (by letter).—After ten years of experimenting the Lower Liao River Conservancy Board has finally succeeded in obtaining about 3 ft. of additional depth in the channel by means of a single jetty across the Liao Bar. The results are praiseworthy. The writer agrees with the author as to the importance of utilizing the flow of the tides for their scouring effect especially since that flow is great enough when guided by a jetty. In the case of the Liao River, the scouring is due mostly to tidal currents rather than to the natural flow of the river. Across this bar, for example, the flow is about 210 000 cu. ft. per sec., which is more than ten times that of the fresh-water discharge from the stream itself.

Furthermore, unless the Upper Liao River is improved generally by the construction of dikes, levees, and other forms of bank protection there is not much hope of controlling the river flow as a scouring medium across the bar. The improvements that would be necessary should be constructed along the well-established principles of good engineering, contrasting in this respect with the present system of permitting the farmers who own the lands along the river banks to construct levees that are almost useless in times of great floods. At present (1930), much of the river flow is naturally impounded on the low flat farm lands in the valley, due to ubiquitous breaks of the poorly located levees mentioned in case of flood. It is then released so gradually that any "flushing" effect that would be brought about by periods of maximum flow is considerably reduced. Under conditions as they exist at present it will require centuries for China to undertake these necessary improvements in the Upper Liao River.

The present Conservancy Board does not have the necessary resources and, therefore, it must continue to depend upon tidal action for the proper maintenance of depth in the navigable channel. Before the relative importance of tidal flow as compared to river flow was generally recognized, members of the Liao River Conservancy Boards were generally of the opinion that all the flood water from the Upper Liao River should be reverted so as to pass the Port of Newchwang. Much of the river flow had previously been by-passed away from Newchwang through the Shuang-tai-tzu Channel. This branch broke through in 1890, eighty miles up stream from Newchwang, and empties into the Gulf of Liao-tung in the vicinity of Pan-shen-hsien (Fig. 10) many miles to the westward. Practically three-fourths of the entire discharge of the river is thus diverted away from Newchwang.

---

[11] Executive Chf. Engr., Upper Liao River Conservancy, Newchwang, China.

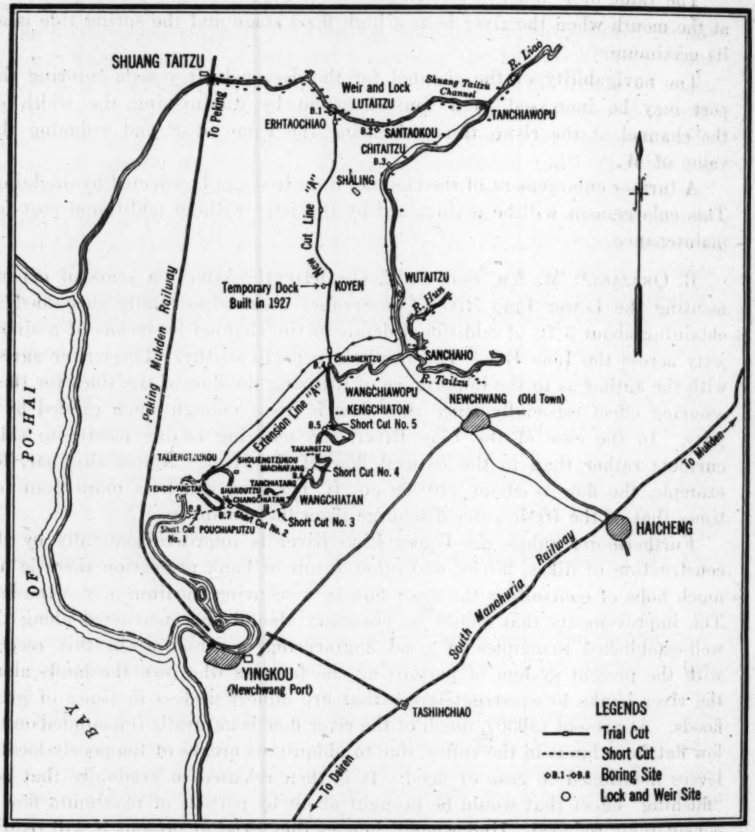

SHUANG TAITZU
To Peking
Weir and Lock
LUTAITZU
Shuang Taitzu Channel
R. Liao
TANCHIAWOPU
B.1
ERHTAOCHIAO
B.2
SANTAOKOU
CHITAITZU
B.3
SHALING
New Cut Line "A"
Peking Mukden Railway
WUTAITZU
Temporary Dock Built in 1927
KOYEN
R. Hun
CHIASHENTZU
B.4
SANCHAHO
R. Taitzu
WANGCHIAWOPU
KENGCHIATON
NEWCHWANG (Old Town)
Extension Line "A"
B.5
Short Cut No. 5
PO-HAI
TAKANGTZU
TALIENGTZUKOU
SHOLIENGTZUKOU
MACHIAFANG
B.6
Short Cut No. 4
To Mukden
TANCHIATANG
TENCHIANGTAI
SHAKOUTZU
CHANGCHIATAN
WANGCHIATAN
B.8
B.7
Short Cut No. 2
Short Cut No. 3
Short Cut No. 1
POUCHIAPUTZU
HAICHENG
South Manchuria Railway
OF
YINGKOU
(Newchwang Port)
BAY
TASHIHCHIAO
To Dairen
LEGENDS
Trial Cut
Short Cut
B.1-B.8 Boring Site
Lock and Weir Site

FIG. 10.—KEY PLAN OF LOWER LIAO BAR.

Handicapped by lack of funds, the Upper Liao River Conservancy Board was obliged to limit the extent of improvement works to the construction of a movable weir across the Shuang-tai-tzu Channel at Erh-tao-chiao, fifteen miles below Tan-chia-wo-pu, although one lock was constructed in connection with this weir and a new canal about fourteen miles long was excavated from Erh-tao-chiao to Chia-shen-tsu. The cross-section of this canal was sufficient only to carry the ordinary low-water discharge of the Upper Liao River. However, with the excavated material built up as levees 400 ft. from the center line on each side, provision was made to meet the demands of part of the flood discharge to be expected.

The object of these improvements by the Upper Liao River Conservancy Board—working in co-operation with the Lower Liao River Conservancy Board as outlined by Mr. Fawcett and the writer—was (1) to recover the inland trade carried by junks; (2) to facilitate navigation in the vicinity of Newchwang in general; and (3) to supplement the scouring force of the tides across Liao Bar with the natural discharge of the river.

At the time these proposed improvements were begun, the writer began making several hydraulic investigations. He soon discovered that, despite the great extent of the drainage area contributing to the river flow, the maximum flood discharge at Tan-chia-wo-pu is only about 60 000 cu. ft. per sec. Comparing this with the 210 000 cu. ft. per sec. of tidal flow at the bar, the relative importance of these two sources of natural power may readily be seen. Subsequently, the Conservancy Board decided to focus its attention on the improvement of the most effective of these two forces rather than to expend the time and money necessary for improving a supply that would be relatively unimportant at best. It was decided that in order to be economical as well as successful from an engineering point of view the improvement of the fresh-water flow would have to await that of the entire river course on a large scale.

TABLE 1.—MONTHLY AVERAGE PERCENTAGE OF SILT, APRIL, 1925, TO MARCH, 1926, INCLUSIVE.

(Figures denote grammes of silt per 100 grammes of water.)

Station and river.	April.	May.	June.	July.	August.	September.	October.	November.	December.	January.	February.	March.	Monthly average.
Liao above Tang-chia-wo-pu....	0.476	0.359	0.599	0.674	0.636	0.234	0.164	0.093	0.027	0.016	0.004	0.011	0.274
Liao below Tang-chia-wo-pu....	0.497	0.371	0.614	0.646	0.623	0.262	0.166	0.108	0.017	0.014	0.004	0.013	0.278
Shuang-tai-tzu below Tang-chia-wo-pu....................	0.503	0.351	0.621	0.622	0.673	0.237	0.154	0.099	0.013	0.017	0.007	0.011	0.276
Liao above San-chia-ho.........	0.381	0.328	0.712	0.633	0.645	0.220	0.150	0.094	0.016	0.025	0.044	0.020	0.272
Liao below San-chia-ho.........	0.191	0.133	0.418	0.357	0.335	0.059	0.069	0.047	0.021	0.032	0.060	0.039	0.147
Tai-tzu-ho at San-chia-ho........	0.085	0.042	0.251	0.238	0.226	0.026	0.032	0.034	0.008	0.015	0.028	0.016	0.083
Shuang-tai-tzu at Erh-tao-chiao.	0.553	0.365	0.749	0.821	0.690	0.266	0.158	0.068	0.024	0.031	0.015	0.039	0.315
Liao at Chia-shin-tzu............	0.211	0.151	0.217	0.256	0.323	0.068	0.059	0.160	0.094	0.045	0.013	0.021	0.135

Aside from the foregoing considerations, there is another factor to be considered in any plan for utilizing the fresh-water flow. The Liao River may be classed as one of the muddiest rivers in the world. During 1925-26 the writer made extensive silt determinations at various places along the Liao River proper. Average determinations for each of the twelve months were made for surface, mid-depth, and bottom elevations. A summary of these results is presented in Table 1, in which the average silt content for the entire section is given at each station. By comparing the silt content of the main or Shuang-tai-tzu River with that of the tributaries (Tai-tzu-ho and Hun-ho) combined, it is seen that the latter carries far less in suspension than the former. Therefore, it is clear that if more water is turned back into the Liao River proper the silt content of this stream would be considerably increased, thus transporting to the bar much more sediment than is being brought by the fresh-water flow and removed by the force of both tidal and fresh-water flow combined, as at present.

In other words, if the fresh-water flow is to be increased for its scouring effect on the bar, this should be done so as to exclude as much of the silt as possible. If an adequate supply of funds could be made available for this purpose it would be advisable to spend it in removing many of the sharp bends and irregularities in the lower reach of the river so as to increase the effect of the tide and to shorten the navigable distance from the Gulf of Liao-Tung to old Newchwang (not new Newchwang where the present port is situated).

The original plan for the improvement of the Upper Liao River is practically completed and that for the lower river, as described by Mr. Fawcett, will be completed in a few years, after obtaining the proper type of dredge for maintenance work. Then the trade of China should be promoted by taking advantage of the benefits derived from works of such magnitude and importance as those of the Upper and Lower Liao River Conservancy Districts. The immediate need is not so much to utilize the river flow as it is to appreciate fully the scouring effect of the tidal currents. This is the most important phase of the problem and should be accomplished with a view to maintaining a channel across the bar by means of dredges and the single training wall (as suggested by the author). Furthermore, the tidal currents should be utilized, with river flow as it is at present; that is, without diverting any more muddy fresh water from up stream (although that may not be in conformity with the author's original idea).

It is desirable that the author submit a method of computing the quantity of silt deposits before selecting equipment for dredging and maintaining a 14-ft. channel alongside and beyond the end of the training wall and also to give an estimate of what fraction of the entire job in view is capable of being done by the small dredge which was recently purchased by the Conservancy. Furthermore, the writer would like to suggest that Mr. Fawcett give the heights of the waves that occur in the vicinity of the bar. If there are no such data available perhaps the author could give some estimates based on his long experience. Is there any information available concerning the probable extent that storms and other sea disturbances would damage the bed of the channel after it has been dredged, especially near the end of the jetty?

P. N. FAWCETT, M. AM. SOC. C. E. (by letter).—Mr. Goodrich has supplemented the paper considerably with particulars of the ports of North China and their respective advantages and disadvantages. His previous association with the Chihli River Commission makes his remarks of particular value. As he states, all the rivers of North China are obstructed by ocean bars. These bars and the very extensive mud-flats, which reach many miles out to sea beyond the coast line, are formed from the deposits of silt brought down from up river during the "freshet" season. Mr. Goodrich states that the percentage of silt in the waters of the Hai River on occasions has been as high as 10, furthermore, the silt is composed of the so-called "loess" soil and is of so fine a nature that it offers considerable difficulty to being removed by drag-suction, hopper dredges because its fineness prevents its settlement in the hoppers.

This class of dredge has been used on the Hai River Bar since about 1914, but the plant is insufficient to obtain a depth of more than 9 ft. at low water, and the maintenance of this depth offers considerable difficulties, so that it is frequently reduced to 6 or 7 ft. during the "freshet" season.

As Mr. Goodrich states, in 1925, the construction of two training walls was undertaken, at the mouth of the Hai River. These walls were completed in 1927, and the work of dredging a channel between them was commenced at the opening of the 1928 season, by means of large bucket dredges. The walls were built to a height of high-water neap tides and extended seaward across the south flat, but only to a point where a low-water depth of 4 ft. naturally obtained. The sites of the two walls at the bend in the Hai River, known as Deep Hole, are shown on Fig. 11. They were to extend about 3.5 miles into the sea beyond Point *C*.

Unfortunately, when dredging was begun, no sill was placed at *Q C* across the Deep Hole where there is a low-water depth of 40 ft., and to make matters worse, the engineers placed a dam across the entrance between the two dikes. This dam was carried to the same height as the dikes, and, therefore, it placed the dredges in a kind of dock. It is evident that if any success was to be obtained, no dam should have been placed across the entrance, but on the other hand, every inducement should have been offered to the ebb currents to enter the cut and assist the dredging. A submerged dam, built to 10 ft. below low water (so as to allow navigation with the same depth as was available across the bar) should have been placed across the Deep Hole at *Q C* (Fig 11), thus throwing as much of the current as possible between the dikes. The dikes should have been extended into deep water and the use of timber should have been avoided.

It was found that after two years intensive dredging, the mud level still remained the same between the dikes as it was outside them, because these methods had not been adopted.

The Conservancy Board, in 1929, called in a Commission of Consulting Engineers, of which the writer was a member, to report upon the project. This Commission—partly because the financial plan resources of the Conservancy were insufficient to pursue the plan further, and partly from a technical viewpoint—recommended the abandonment of the plan and the pulling up of the dikes.

In the writer's opinion, the dikes were incorrectly located, the present natural Deep Hole Channel should have been utilized and trained and carried out in its continuation across the bar. As it is, more than $1 000 000 (gold) was spent on this plan, all of which was a total loss.

Mr. Goodrich has always felt that a single curved jetty across the Hai Bar might have provided the desired benefits. The writer concurs with this opinion, and such a wall was designed and proposed by Mr. A. de Linde, as long ago as 1902, while he was Engineer-in-Chief of the Hai River Commission.

Dr. Chatley gives some interesting data of the Whangpoo River and describes results obtained by the single training wall originally constructed at its mouth, which was afterward supplemented by an additional wall. In this particular locality, the single wall increased the depth across the bar from 15 to 21 ft., and the double wall further increased this depth to 30½ ft. He is of the opinion that the results obtained on the Whangpoo River show that the scour induced by a single training wall can be usefully supplemented by a double wall.

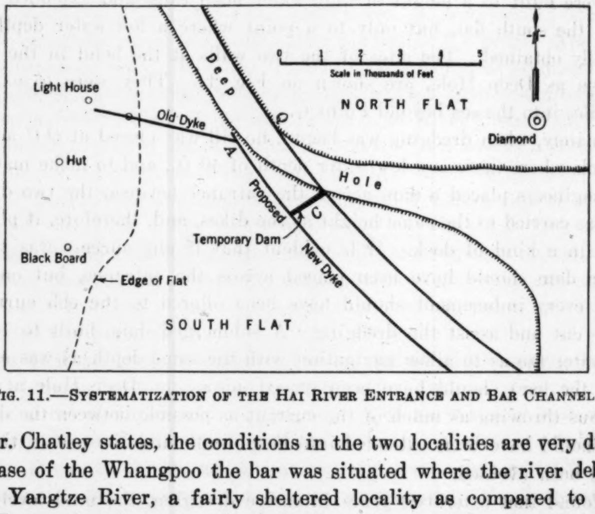


FIG. 11.—SYSTEMATIZATION OF THE HAI RIVER ENTRANCE AND BAR CHANNEL

As Dr. Chatley states, the conditions in the two localities are very different. In the case of the Whangpoo the bar was situated where the river debouched into the Yangtze River, a fairly sheltered locality as compared to the sea coast. Every situation must be judged on its merits and each of them presents its own particular problems. In the case of the Liao Bar, where the ebb and flow of the tidal currents take place in the directions described in the paper, and where the bar is situated eight miles from the river entrance, the writer is of the opinion that the construction of a second training wall would necessitate too great a risk that the expenditure involved would not be justified by the increase in the depth that might or might not result. Added to this, and partly owing to the intensive ice action, a very high annual maintenance charge would need to be added to the capital cost. It was with these points in view that the writer limited the construction to a single wall.

Mr. Ripley considers that the increase in depth across the Liao Bar is caused by overfall of the flood tide. This, however, is not the case, there being practically no overfall, the flow of the flood tide taking place very quietly and the surface of the water being at the same level on both sides of the wall. The writer is, of course, familiar with Mr. Ripley's proposed method of deepening a bar by beginning the construction from the seaward end, and working inshore, but considers that it would be difficult to persuade commercial bodies to sanction such a method of work.

Dr. Okazaki is quite correct in pointing out the difficulties of improving the flow of the Upper Liao River owing to the refusal of the Chinese authorities to take any constructive interest in matters of conservancy. Had funds been forthcoming, the training wall across the Liao Bar could have been completed in two instead of ten years. Furthermore, it is probable that a greater improved depth would have been obtained, since the alteration of the surface slopes would have taken place more suddenly.

The writer agrees with Dr. Okazaki that the tidal flow is of the greatest importance in obtaining and maintaining a channel across the bar. He does not concur, however, with the view expressed, that the improvement of the fresh-water flow should not be undertaken pending the completion of works on the bar. Far from this, the records of this locality show that the deterioration of the bar dated from about the time when the Liao River was diverted down the Shuang-tai-tzu River. Furthermore, since that date the entire estuary of the Liao has become encumbered with sand-banks and islands which are accreting with great rapidity and thus excluding the tidal influx, so that it is only a matter of time—unless adequate steps are taken—when the estuary will be filled up.

It is a well-known axiom that "tidal rivers cannot be maintained without their fresh-water flow." It was with the intention of securing its original fresh-water discharge back to the Liao Estuary that works were built by the Conservancy Board in the upper river, and the weir and cutting described by Dr. Okazaki were constructed at a cost of approximately $700 000 (gold).

In spite of the writer's continued warnings, these works are not being utilized to their full extent, and practically no reversion of fresh water has occurred since they were completed. Unfortunately, the Board has no control over the area, as it is under Chinese jurisdiction. In the case in point, instead of causing the cutting to be scoured to its full width of 800 ft. and the fresh water reverted past Newchwang, the cut is maintained at a width of about 100 ft., and the remainder of the land between the embankments is leased by the officials for farming in spite of the fact that this land was bought by the Conservancy Board.

Works built up country in China are also subject to much damage and interruption by the local population and banditti, and a weir constructed across the Shuang-tai-tzu River in 1912 to re-divert the flow back into the Liao was destroyed by the villagers as soon as it was completed, because it interfered with the lucrative practice of salt smuggling.

Dr. Okazaki believes that it is better for the estuary and bar if this fresh-water discharge is allowed to continue to escape down the Shuang-tai-tzu River as it carries an extremely high silt content, whereas the tributaries, the Tai-tzu and the Hun, are comparatively siltless. The writer's views are entirely the opposite, and he would point out that the silt of the Shuang-tai-tzu is of so extremely fine a character that it is all carried completely across the bar into deep water at sea before settlement occurs. The Hun River and Tai-tzu River, on the other hand, show clean and clear fresh-water discharges; yet, in times of heavy rains, these two rivers roll large quantities of extremely heavy sand along the bottom, which, on one occasion, completely silted the Newchwang Estuary, to a depth of 12 ft.

During this "freshet" samples taken for silt content, 1 ft. above the river bed, gave no indication of this sand. In view of this, the silt table (Table 1) presented by Dr. Okazaki would give no indication of the true nature of the problem.

The fineness of the Upper Liao silt is somewhat remarkable as shown in Table 2. In this study, 51% of the Liao soil was found to have a grain diameter of less than 0.050 mm.

In writing this paper to describe the effect on the Liao Bar of constructing the single training wall, the writer felt that particular interest would attach to the effects, considering the fact that no dredge had been used in collaboration with the construction of the training wall. The reason for this was entirely due to the fact that funds available were not sufficient for its purchase.

In 1929, the author was enabled to purchase the drag-suction hopper dredge, *Java,* from the Dutch East Indies Government. This vessel, built in 1911, is fitted with twin pumps and propellers driven by engines of 560 and 900 i.h.p., respectively. The suction pipe is 37½ in. in diameter and the hopper of 750 cu. yd. capacity.

TABLE 2.—COMPARISON OF FINENESS OF SILT

Item	Ymuiden	Seine	Rotterdam Waterway	Hai-Ho	Liao-Ho
Percentage passed through sieve of 4 900 meshes per square centimeter..........	2	23	45	48	94
Mean size of grain, in millimeters*.......	0.167	....	0.102	....	0.053

* After method described by Messrs. T. Heyblom and A. G. Maris, Paper 76, XIV Inter. Cong. of Navigation.

Dredging was begun in April, 1930, and in spite of many interruptions, due to the enormous quantities of iron bars, wire rope, Chinese anchors, etc., that kept interrupting the work, a channel 12 ft. deep and 200 ft. wide had almost been completed by the end of September. Freshets then brought down large quantities of sand from the Tai-tzu and Hun Rivers, with the result that the dredged cut almost became obliterated.

With the Shuang-tai-tzu fresh-water discharge returned to the Liao Estuary the writer is of the opinion that the damaging sand deposits of the Tai-tzu and Hun Rivers would be flushed across the bar and deposited out at sea. The situation is very similar to that at Tientsin to which Mr. Goodrich calls attention, where disaster is caused to the improvement works in the lower river and harbor, by the neglect that has been allowed, of the upper portions of the rivers and tributaries. The writer is, therefore, not in agreement with Dr. Okazaki and considers that the permanent improvement of the bar is impossible, unless the tributaries and upper river are first put in order.

As regards the dredging plant required, a 14-ft. channel across the bar should be obtainable with the dredge, *Java*, as long as no further excessive sand deposits occur.

The writer does not consider that the height of the waves ever exceed 10 ft., and no flattening of the channel has ever been experienced after storms. On the other hand, at the Hai River Bar, the channel is beaten down and reduced in depth after each severe storm. The failure of such storms to flatten the Liao Bar, the writer believes, is entirely due to the protection offered to the channel by the training wall, which receives the force of the waves on its convex side.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

TRANSACTIONS

Paper No. 1782

EVAPORATION AS A FUNCTION OF INSOLATION[1]

By Burt Richardson,[2] Esq.

With Discussion by Messrs. H. H. Kimball, C. G. Abbot, D. P. Krynine, R. C. Wells, N. W. Cummings, and Burt Richardson.

Synopsis

In the Western States water is scarce and valuable, and rates of evaporation are high. Heretofore, evaporation losses have been ascertained by direct measurements. The writer considers it possible to approach the problem by a study of the physical cause of evaporation. The principal cause is insolation which may be defined, briefly as "exposure to the rays of the sun."

This paper is concerned with the effect of insolation and with the measurement of the quantity of radiant heat imparted by the sun and sky to the water and the bottom of a reservoir. In a sense, this effect may be considered a measure of insolation. For example, it may be said that, in Pasadena, Calif., on a sunny day, the "insolation" is roughly 600 calories per sq. cm. per day.

Introduction

The quantitative effect of insolation is variable and depends on the sun's elevation, the length of the day, cloudiness, smoke, etc. It may be determined by three independent methods:

(1) By means of a Weather Bureau, recording, thermo-electric pyrheliometer.
(2) By computing the quantity as a geometric problem from a knowledge of the sun's radiation, altitude, etc., together with the latitude of the reservoir or lake.
(3) By tracing the radiant heat energy which strikes the water surface of a heat-insulated tank.

Note.—The Special Committee on Irrigation Hydraulics selected the subject of "Evaporation Losses from Reservoirs" as one of ten for study and research. This paper was submitted to the Committee by its author, and the Committee recommended its publication (see Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 176).

[1] Published in May, 1930, *Proceedings*.

[2] Prof., Dept. of Physics, Occidental Coll., Los Angeles, Calif.

Having determined the value of insolation for a particular site it is possible to show what part of it goes into evaporation and to place definite limits on the quantity of evaporation. By using either Method (1) or Method (2) in combination with Method (3) the evaporation from a lake or proposed reservoir may be determined. Attention is called to the fact that there is a regrettable scarcity of reliable data for evaporation studies. The data desirable for such studies include:

(*a*) Daily, monthly, and yearly records of evaporation from lakes and pans all observed in a uniform manner.
(*b*) Insolation records, especially in country districts and in the Western States (the present (1929), six U. S. Weather Bureau Stations at which insolation records are secured, are located in cities).
(*c*) Records of the temperature gradient of lakes.
(*d*) Cloudiness records kept in a uniform manner (at present, there is no unit for cloudiness, and it is impossible to convey to another the meaning of a cloudy or very cloudy day).
(*e*) Records of transmission coefficients of the atmosphere for various sections of the United States to ascertain if there exists a "mean weighted transmission coefficient" for a certain district.
(*f*) Data concerning water surface and air temperatures of lakes to provide a means of computing radiation, which is a factor in the evaporation equation.

NOTATION

The notation adopted for use in this paper is as follows:

A = correction coefficient = $\dfrac{\text{solar radiation}}{\text{total radiation}}$.
a = the radiation constant introduced by Stefan-Boltzmann.
B_r = the radiation from water.
b = a constant which permits the use of the air temperature above a body of water instead of the sky temperature to which the radiation actually goes.
C = a factor representing the conduction of heat through the walls of the tank.
C' = the percentage of cloudiness that occurred during a certain month, as given in *Monthly Weather Review* reports.
d = the sun's declination for a certain day of the year.
E = quantity of water evaporated.
F = factor denoting the relative clearness of the sky above the particular body of water in question.
H = original vapor pressure of the air passing over the surface of the water.
i = angle of incidence.
I = the quantity of radiant heat, in calories per minute, that reaches a certain area on the surface of the earth.
I_{ex} = insolation incident upon the exterior of the earth's atmosphere.
K = the solar constant.
k = ratio of the incident to the reflected intensity of radiation.
L = latent heat of evaporation.
n = index of refraction.
P = atmospheric pressure.
P_w = original vapor pressure of the air contact with the water surface.
Q = the observed radiant energy that passes through local clouds.
Q_s = the observed radiant energy that passes through a clear sky.

R = the ratio convection evaporation.

r = angle of refraction.

S = the sensible heat measured by the warming or cooling of the water.

T_a = original temperature, centigrade, of the air passing over the surface of the water.

T_w = original temperature, centigrade, of the air in contact with the water surface.

t = time, in minutes.

$w = \frac{2\pi}{24 \times 60}$ radians per min. = angular velocity of the earth on its axis.

λ = the latitude of the station.

ρ = the transmission coefficient of the earth's atmosphere.

ρ' = the mean transmission coefficient of the atmosphere at a particular place.

ψ = the zenith angle of the sun.

(1) INSOLATION RECORDED BY THERMO-ELECTRIC PYRHELIOMETERS

The Weather Bureau of the U. S. Department of Agriculture maintains six stations at which recording thermo-electric pyrheliometers[3] are used and their records of insolation are published in the *Monthly Weather Review*. Due to the difference in latitude, cloudiness, etc., the value of insolation is, of course, not the same for all stations. For example, the average number of calories reaching each square centimeter of the earth's surface per day during the month of July, 1928, at certain places is indicated in Table 1.

TABLE 1.—VALUES OF INSOLATION AT SEVEN STATIONS IN THE UNITED STATES.

Location.	Insolation, in calories per square centimeter per day.	Location.	Insolation, in calories per square centimeter per day.
New York, N. Y.	378.5	Lincoln Nebr.	531.2
Washington, D. C.	540.8	Twin Falls, Idaho	737.2
Chicago, Ill.	412.8	Scripps Institute of Oceanography, La Jolla, Calif.	544.7
Madison, Wis.	509.2		

The insolation varies also from day to day at any one station, as illustrated by data observed at the Scripps Institute of Oceanography at La Jolla, Calif. While this is not a U. S. Weather Bureau Station, it uses the U. S. Weather Bureau Pyrheliometer No. 17.

Observations for each day of July, 1928, are given in Table 2. The total insolation for the 31 days was 16 886.3 calories, which is the equivalent of 10.35 in. of evaporation, or an average of 544.7 calories per day.

Considering that it requires 585.4 calories to evaporate 1 cu. cm. of water at 20° cent., it is easily seen how much a given water surface would drop each day due to evaporation, should all the insolation go into evaporation. Actually, the ratio of observed evaporation to observed insolation is between 40 and 60 per cent.

Table 3 gives all the records of insolation that have been observed for stations in the United States. It has been compiled from various sources. Items

---

[3] H. H. Kimball and H. E. Hobbs, *Journal*, Optical Soc. of America, Vol. 7, September, 1923.

Nos. 1 to 18, inclusive, are records at six stations of the U. S. Weather Bureau where insolation is observed;[4] Items Nos. 19 to 42, inclusive, are records reported from sundry other stations;[5] and the observations listed for the last five stations are those observed by the writer, using the pan method.[6]

TABLE 2.—INSOLATION AS RECORDED BY THE U. S. WEATHER BUREAU PYRHELIOMETER NO. 17, AT SCRIPPS INSTITUTE OF OCEANOGRAPHY, LA JOLLA, CALIF.

Date, July, 1928.	Insolation, in calories per square centimeter per day.	Date, July, 1928.	Insolation, in calories per square centimeter per day.
1	662.2	16	600.6
2	627.1	17	604.1
3	669.1	18	579.6
4	679.1	19	632.3
5	620.6	20	481.9
6	614.0	21	584.2
7	610.3	22	432.8
8	591.8	23	358.6
9	665.9	24	275.5
10	605.1	25	238.0
11	586.5	26	517.4
12	471.9	27	592.4
13	482.9	28	489.2
14	633.6	29	474.7
15	571.4	30	410.0
		31	579.0

(2) INSOLATION COMPUTED AS A GEOMETRIC PROBLEM

It is possible to determine the value of insolation from a knowledge of the sun's radiation, its altitude, etc., and the latitude of the reservoir or lake. A formula may be set up for insolation, as follows:

$$I = K \int_{t_1}^{t_2} \cos \psi \, \rho^{\sec \psi} \, d\,t \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Differentiating with respect to time to obtain the rate of insolation,

$$\frac{d\,I}{d\,t} = K \cos \psi \, \rho^{\sec \psi} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The solar constant, K, as determined by Abbot,[7] is 1.93 calories per sq. cm. per min. To illustrate the use of Equation (2) consider the parallel of 40° N. latitude. Use Abbot's value for the solar constant ($K = 1.93$); for simplicity assume a transmission coefficient of $\rho = 0.7$, and insert proper values of ψ for each minute of each day of each month, successively. The values of I thus obtained are given in Table 4.

At this point something must be said about ρ, the transmission coefficient[7] of the earth's atmosphere used in Equation (2). The only available values of this coefficient are as shown in Table 5. These were weighted values com-

---

[4] "Average of Values for a Particular Month Printed in *Monthly Weather Review* from Date of First Observation at a Station until January, 1929," *Monthly Weather Review*, Vol. 57, January, 1929.

[5] *Monthly Weather Review*, October, 1928, p. 430.

[6] National Research Council, *Bulletin No. 68*, April, 1929, and *Monthly Weather Review*, July, 1929, p. 300.

[7] *Annals*, Astrophysical Observatory of the Smithsonian Institution, C. G. Abbot, Vol. 4, 1922.

TABLE 3.—THE MEAN MONTHLY QUANTITIES OF OBSERVED INSOLATION. ALSO, THE MEAN MONTHLY INSOLATION COMPUTED BY EQUATION (12) INCIDENT UPON THE EXTERIOR OF THE EARTH'S ATMOSPHERE. THE RATIO OF THE OBSERVED INSOLATION TO THAT COMPUTED ON THE EXTERIOR OF THE EARTH'S ATMOSPHERE.

Item No.	Station.	Position.		Nature of data.	Insolation, in Calories per Square Centimeter per Day.											
		North latitude.	West longitude.		January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.
1	New York, N. Y.	40°-46′	....	Observed	128	235	262	354	374	358	402	276	264	195	114	91
2				Computed	373	560	650	793	964	1 000	1 012	888	700	518	377	297
3				Ratio	0.343	0.42	0.404	0.446	0.388	0.358	0.397	0.311	0.377	0.377	0.303	0.306
4	Washington, D. C	38°-56′	....	Observed	178	242	338	419	472	489	473	420	360	279	188	143
5				Computed	377	577	678	820	994	1 020	1 022	894	705	548	410	326
6				Ratio	0.472	0.42	0.50	0.51	0.475	0.483	0.468	0.47	0.51	0.51	0.46	0.44
7	Chicago, Ill.	41°-47′	....	Observed	98	151	223	328	432	335	390	380	336	167	87	78
8				Computed	317	547	643	785	961	1 019	1 036	890	683	510	363	286
9				Ratio	0.31	0.278	0.347	0.418	0.45	0.320	0.375	0.427	0.493	0.328	0.24	0.273
10	Madison, Wis.	43°-05′	....	Observed	165	245	315	414	450	519	505	434	334	221	139	125
11				Computed	303	534	629	777	949	1 008	1 085	891	684	524	357	279
12				Ratio	0.545	0.46	0.50	0.532	0.475	0.519	0.465	0.487	0.49	0.421	0.39	0.45
13	Lincoln, Nebr.	40°-50′	....	Observed	208	300	374	451	475	556	570	486	392	290	203	171
14				Computed	324	561	650	794	965	999	1 020	889	690	515	379	298
15				Ratio	0.643	0.534	0.575	0.568	0.492	0.556	0.564	0.548	0.568	0.563	0.536	0.573
16	Twin Falls, Idaho	42°-27′	....	Observed	162	261	250	345	380	614	717	708	567	390	180	150
17				Computed	308	550	647	787	954	1 018	1 024	887	688	504	354	280
18				Ratio	0.526	0.475	0.387	0.44	0.40	0.60	0.70	0.798	0.823	0.774	0.51	0.536
19	Vancouver, B. C., Canada	52°-0′	129°-0′	Observed	81	147	204	283	366	357	326	304	199	131	74	53
20				Computed	196	342	549	792	943	1 006	942	791	571	347	196	153
21				Ratio	0.41	0.43	0.37	0.36	0.39	0.35	0.35	0.38	0.35	0.38	0.38	0.35
22	Near Victoria, B. C , Canada	48°-0′	124°0′	Observed	94	157	259	339	414	405	388	335	255	153	90	71
23				Computed	249	396	592	815	957	1.011	954	818	618	408	258	202
24				Ratio	0.38	0.40	0.44	0.42	0.43	0.40	0.41	0.41	0.41	0.38	0.36	0.35
25	Near St. Johns, N. B., Canada	48°-0′	60°-0′	Observed	114	206	270	345	397	424	422	364	284	165	106	78
26				Computed	249	396	502	815	957	1 011	954	818	618	408	254	202
27				Ratio	0.46	0.52	0.46	0.42	0.42	0.42	0.44	0.45	0.46	0.41	0.43	0.39

TABLE 3.—(*Continued*).

Item No.	Station.	Position.		Nature of data.	Insolation, in Calories per Square Centimeter per Day.											
		North latitude.	West longitude.		January.	February.	March.	April.	May	June.	July.	August.	September.	October.	November.	December.
28	Boat off Boston, Mass.	42°-0′	66°-70′	Observed	139	228	327	402	449	477	421	376	317	235	153	120
29				Computed	331	477	655	848	964	1 014	965	850	678	475	332	282
30				Ratio	0.42	0.47	0.50	0.47	0.47	0.47	0.44	0.44	0.47	0 50	0.46	0.43
31	Near Eureka, Calif.	42°-0′	124°-0′	Observed	148	246	318	435	486	524	442	382	318	233	151	129
32				Computed	331	477	655	848	964	1 014	965	850	678	475	332	282
33				Ratio	0.45	0.52	0.48	0.51	0.51	0.52	0.46	0.45	0.47	0.49	0.46	0.46
34	Lower California, Mexico	30°-0′	117°-0′	Observed	306	364	415	482	476	482	449	437	420	340	301	281
35				Computed	494	615	761	898	978	997	978	898	774	618	496	453
36				Ratio	0.62	0.59	0.55	0.54	0.49	0.48	0.46	0.49	0.54	0.55	0.61	0.62
37	Boat off Florida	30°-0′	65°-77′	Observed	212	247	365	420	462	441	432	399	326	259	244	197
38				Computed	491	615	761	898	973	997	978	898	774	618	496	458
39				Ratio	0.43	0.40	0.48	0.47	0.47	0.44	0.44	0.44	0.42	0.42	0.49	0.43
40	Santiago, Cuba	20°-0′	77°-0′	Observed	335	420	446	475	452	493	464	450	428	379	324	307
41				Computed	624	728	832	922	950	957	950	920	840	726	621	589
42				Ratio	0.54	0.58	0.54	0.52	0.48	0.52	0.49	0.49	0.51	0.52	0.52	0.52
43	Pasadena. Calif	34.13°	....	Observed	302	383	427	532	568	512	582	568	490	400	365	316
44				Computed	430	545	704	849	960	1 011	986	902	779	616	474	418
45				Ratio	0.70	0.70	0.61	0.63	0.59	0.51	0.59	0.63	0.63	0.65	0.76	0.75
46	Fort Collins, Colo.	40.58°	....	Observed	......	......	......	......	......	...	......	......	370	......	......	......
47				Computed	......	......	......	......	......	......	......	......	647	......	......	......
48				Ratio	......	......	......	......	......	......	......	......	0.57	......	......	......
49	San Francisco, Calif	37.46°	....	Observed	......	......	......	......	......	......	......	564	......	......	......	......
50				Computed	......	......	......	......	......	......	......	874	......	......	......	......
51				Ratio	......	......	......	......	......	......	......	0.645	......	......	......	......
52	San Diego, Calif	32.75°	....	Observed	......	......	......	......	......	......	662	......	......	......	......	......
53				Computed	......	......	......	......	......	......	955	......	......	......	......	......
54				Ratio	......	......	......	......	......	......	0 70	......	......	......	......	......
55	La Jolla, Calif.	32.79°	....	Observed	264	316	365	433	487	501	545	438	285	291	298	275
56				Computed	......	......	......	......	......	......	967	......	......	......	......	......
57				Ratio	......	......	......	......	......	......	0.564	......	......	......	......	......

puted from the mean solar intensity and coefficient for a particular wave length.

Values of the transmission coefficient for other places in the United States have not been computed so that research on this coefficient is highly desirable. The writer suggests that this coefficient may be computed directly by means of Equation (2). The left side of this equation which represents the rate of insolation is known from the pyrheliometer record by direct reading for a particular minute of the day, and the zenith angle[8] corresponding to this minute is also known, which leaves the one unknown, ρ.

TABLE 4.—COMPUTED VALUES OF I AT 40° NORTH LATITUDE.

Month.	Values of I, in calories per square centimeter per day.	Month.	Values of I, in calories per square centimeter per day.
January	140.0	July	553.3
February	244.2	August	457.3
March	356.5	September	347.7
April	465.7	October	240.0
May	573.7	November	136.0
June	586.7	December	100.1

Total energy, including scattered sky radiation, is recorded by a pyrheliometer. Therefore, it is necessary to correct the pyrheliometer records by a coefficient, A, which represents the ratio of solar radiation to total radiation. Typical values of A for four stations[9] are given in Table 5.

TABLE 5.—OBSERVED VALUES OF ρ AND A.

Station.	North latitude.	Values of ρ.	VALUES OF A FOR DIFFERENT ZENITH ANGLES, ψ.						
			0°	7.5°	25°	30°	48.3°	55°	60°
Mt. Wilson, Calif.	34°-13′	0.841	87	86	86	86	84	80	80
Washington, D. C.	38°-56′	0.689	87	85	84	84	83	81	80
Mt. Whitney, Calif.	......	0.882	......	......	......	......	......	......	......
Madison, Wis.	40°-51′	......	87	86	85	85	84	82	81
Lincoln, Nebr.	43°-05′	......	87	86	84	84	84	79	76

The rate of insolation from the sun (not including sky radiation) equals:

$$\frac{d\,I'}{d\,t} = A\,\frac{d\,I}{d\,t} = K \cos\psi\,\rho^{\sec\psi} \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and, making the necessary transpositions:

$$\rho = \log^{-1}\left[\frac{\log\left(\dfrac{A\,\dfrac{d\,I}{d\,t}}{K\cos\psi}\right)}{\sec\psi}\right] \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

---

[8] "American Practical Navigator," Bowditch.

[9] *Monthly Weather Review*, April, 1927, p. 156.

To illustrate this method for computing the transmission coefficient, the pyrheliometer records of fourteen different days of 1926 for Pasadena, Calif., were used, and when monthly values were plotted, a mean value for ρ of 0.751 was obtained.

The value of insolation is not constant along any parallel of latitude, but changes at points of different longitude, due to variation in the transmission coefficient, humidity, local clouds, smoke, dust, etc. Therefore, in order to use Equation (2) for points of different longitude, values of the transmission coefficient must be observed by Abbot's method or computed by Equation (4).

(3) INSOLATION BY TRACING RADIANT HEAT ENERGY

The third method[10] for measuring insolation is to trace the heat energy which comes to water contained in an open tank. In this case insolation (when corrected for reflection) will be equal to the summation of: (*a*) Evaporation; (*b*) convection losses, due to the circulation of air above water; (*c*) sensible heat, measured by the warming of the water; (*d*) conduction, due to loss of heat through the walls of the tank; and (*e*) radiation, from the water to colder air. This is simply a statement of the Law of Conservation of Energy.

The ratio, *k*, of the incident to the reflected intensity of radiation is given by Fresnel's equation:

$$k = \frac{1}{2}\,\frac{\sin^2 (i-r)}{\sin^2 (i-r)} - \frac{1}{2}\,\frac{\tan^2 (i-r)}{\tan^2 (i-r)} \ldots\ldots\ldots\ldots\ldots (5)$$

in which, i and r are the angles of incidence and of refraction, respectively, and are connected by the relation, $n = \frac{\sin i}{\sin r}$ = index of refraction.

Let n, for water, be equal to 1.33; then r may be found for corresponding values of i and these, substituted in Equation (5), will give values of k for any angle of incidence. This is the correction factor for reflection.

Insolation is a positive quantity during the day, but at night it is equal to zero. Evaporation is always positive, while the other terms—convection, sensible heat, conduction, and radiation—may be either positive or negative. In symbols, then:

$$I = LE + LER + S + C + B_r \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which, L is the latent heat of evaporation (585.4 at 20° cent.); E is cubic centimeters of water evaporated; and R is the ratio, $\frac{\text{convection}}{\text{evaporation}}$.

Bowen[11] has found that:

$$R = \frac{0.46\,(T_w - T_a)\,P}{(P_w - H)\,760} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which, T_a and H are the original temperature and vapor pressure of the air passing over the surface of the water; T_w and P_w are the corresponding quantities for the layer of air in contact with the water surface; P is the atmospheric pressure; S is the sensible heat measured by the warming or cooling of the water; and C represents the conduction of heat through the walls of the tank.

---

[10] N. W. Cummings and Burt Richardson, *Physical Review*, October, 1927.

[11] *Physical Review*, June, 1926.

B_r is the radiation from water[12] and may be computed by the formula:

$$B_r = (0.906\, a\, T_w^4 - b\, a\, T_a^4) \ldots\ldots\ldots\ldots (8)$$

in which, the decimal, 0.906, represents an important constant which takes care of the fact that water does not radiate as a perfectly black body; a is the Stefan-Boltzmann radiation constant, amounting to 5.7×10^{-5} ergs per sq. cm. per sec. per degree to the fourth power; while b permits the use of air temperature in the formula in place of temperature of the sky which receives the radiant heat energy. The average value of the constant, b, in the radiation Equation (8) has been observed at four stations, to be as follows:

Station.	Mean value of b.
Tank at Pasadena, Calif.	0.815
Tank at Fort Collins, Colo.	0.760
Murray Lake, San Diego, Calif.	0.794
Crystal Springs Lake, San Francisco, Calif.	0.757
Average (weighted)	0.790

When the temperature of the water and air is known, by using the mean value of 0.790 for b, radiation may be computed by Equation (8). However, when these temperatures were not known, it was possible, in the case of California water surfaces, to estimate the value of radiation from the data collected at other places. The radiation from the water to the sky, in calories per square centimeter per day, has thus been found for five places as shown in Table 6.

TABLE 6.—RADIATION FROM THE WATER TO THE SKY, IN CALORIES PER SQUARE CENTIMETER PER DAY.

Location.	Radiation.
Pasadena, Calif.	117.0
Fort Collins, Colo.	132.0
Murray Lake, San Diego, Calif.	128.0
Crystal Lake, San Francisco, Calif.	148.0
La Jolla, Calif.	130.3
Average value (weighted)	131.4

From Equation (6) the exact amount of radiation may be determined for the night, since insolation during the night is zero and all other quantities of the equation are measurable. Thus, the insolation may be determined by adding all the heat energy coming to the body of water. Table 7 demonstrates the order of magnitude of each item that enters into insolation. A similar tabulation for observations at Murray Lake, San Diego, Calif. (July 7 to July 27, 1927), gave corresponding average daily values, as follows:

---

[12] National Research Council, *Bulletin No. 68*, April, 1929.

Factors.	Calories per square centimeter per day.
Evaporation	485.9
Sensible heat	0.8
Convection	34.4
Back radiation	128.2
Conduction	13.0
Observed insolation	662.3

TABLE 7.—ITEMS THAT MAKE UP PAN INSOLATION, AS OBSERVED AT CRYSTAL SPRINGS LAKE, SAN FRANCISCO, CALIF., AUGUST 10-30, 1927.
(In calories per square centimeter per day.)

Day.	Evaporation.	Sensible heat.	Convection.	Back radiation.	Conduction.	Pan observed insolation.
1	389.1	0	103.2	147.1	19.3	658.7
2	232.5	— 1.9	40.5	145.5	17.6	434.2
3	438.9	— 28.0	51.1	141.3	12.5	615.8
4	403.0	13.0	6.7	144.5	9.6	576.8
5	351.8	83.7	26.0	148.0	14.4	623.9
6	494.2	09.4	101.4	171.4	30.6	707.0
7	355.5	— 9.3	94.5	165.0	27.6	633.3
8	326.4	— 3.6	87.7	162.5	26.2	599.2
9	395.3	33.5	66.6	158.8	20.2	674.4
10	412.2	— 11.2	14.3	139.0	10.5	564.8
11	393.4	0	— 8.0	144.0	2.8	532.2
12	392.5	063.2	19.2	136.2	10.9	622.0
13	327.4	48.4	10.5	139.0	11.1	536.4
14	311.5	— 5.5	22.6	146.5	13.1	488.2
15	381.8	— 13.0	15.3	145.8	13.1	543.0
16	370.2	— 9.4	29.8	141.5	11.2	543.3
17	315.0	7.5	65.4	150.2	15.1	553.2
18	243.8	— 40.8	33.6	136.1	8.7	381.4
19	383.4	63.3	19.5	139.6	8.2	614.0
20	379.2	26.1	24.6	142.5	12.1	584.5
Total, 20 days	7 197.1	225.4	824.5	2 944.5	294.8	11 486.3
Average per day	359.9	11.3	41.2	147.2	14.7	574.3

As a test of this method, tanks of different areas and depths have been measured hourly over long periods of time with a resulting difference of less than 3 per cent. As one illustration, Table 8 will show a comparison of the results obtained from three tanks of different areas and depths and a Weather Bureau pyrheliometer (No. 17) over a period of five days, at the Scripps Institute of Oceanography, La Jolla, Calif.

TABLE 8.—COMPARISON OF INSOLATION QUANTITIES OBSERVED BY DIFFERENT METHODS.

Method of observation.	INSOLATION, IN CALORIES PER SQUARE CENTIMETER PER DAY.					Average.
	July 9.	July 10.	July 11.	July 12.	July 13.	
Pyrheliometer	662.5	614.2	583.0	468.2	476.2	560.8
Tank No. 1	653.6	617.7	594.0	463.4	496.3	565.0
Tank No. 2	629.0	590.3	565.2	445.9	480.7	542.2
Tank No. 3	688.3	626.7	578.9	474.5	443.4	552.3

DESCRIPTION OF METHOD FOR MEASURING EVAPORATION

By transposing Equation (6) which is in metric units, and converting to inches, it may be written:

$$E = \frac{I - S - C - B_r}{2.54\ L\ (1 + R)} \quad \text{..........................} \quad (9)$$

This expression, which gives evaporation as a function of insolation, water temperature, wet and dry bulb temperatures, and vapor and atmospheric pressures, is a true equation; but the tedious process of evaluating each term hourly (as was done in the case of data mentioned in this paper) would defeat the purpose of this method for practical use. As a result of research, it is possible to assign limiting values to each term of Equation (9) and by considering certain variables constant, an equation may be obtained which is simple to handle. Thus, by considering sensible heat and conduction negligible, using 585 for the latent heat of evaporation, and 0.22 for Bowen's ratio, R, Equation (9) becomes, in inch units:

$$E = \frac{I - B_r}{2.54 \times 585 \times 1.22} = \frac{I - B_r}{1\,814} \quad \text{...............} \quad (10)$$

Thus far, no mention has been made of the effect of wind upon evaporation, because in the expression for insolation given by Equation (6) this effect would be measured, calorie for calorie, in changes in sensible heat and in convection. Sensible heat as well as the value of conduction for a short period or over a yearly cycle, as determined from a study of data collected in California, is negligible. Furthermore, no serious error will result from considering R as a constant equal to 0.22. This may be seen by referring to Table 9 which gives the values of R computed under dissimilar circumstances and over a long period of time with hourly readings.

TABLE 9.—COMPARISON OF VALUES OF R COMPUTED UNDER DISSIMILAR CIRCUMSTANCES.

Location.	Date.	Interval, in days.	Ratio, R.
Pasadena, Calif.	**June, 1926**	**10**	**0.193**
Fort Collins, Colo.	**September, 1926**	**10**	**0.221**
Murray Lake, San Diego, Calif.	**July, 1927**	**20**	**0.165**
Crystal Springs Lake, San Francisco, Calif.	**August, 1927**	**20**	**0.280**
Scripps Institution Pier, La Jolla, Calif.	**July, 1928**	**5**	**0.180**
Pacific Ocean, Santa Babara Channel, Calif.	**July, 1928**	**12**	**0.280**
U. S. Naval Air Station: San Diego Bay, Calif.	**August. 1928**	**31**	**0.190**
San Diego Bay, Calif.	**September, 1928**	**30**	**0.208**
San Diego Bay, Calif.	**November, 1928**	**30**	**0.171**
San Diego Bay, Calif.	**March, 1929**	**31**	**0.170**
San Diego Bay, Calif.	**April, 1929**	**30**	**0.152**

The maximum deviation in these values of R from the mean is 0.06, and since evaporation equals 50% of insolation on the average, the resulting error in insolation caused by taking R equal to a constant, 0.22, instead of the largest or smallest value of R computed, would be 3 per cent.

TABLE 10.—OBSERVED EVAPORATION AT THIRTY-TWO STATIONS IN THE UNITED STATES, IN INCHES PER MONTH.

Station.	Relative location, in degrees north latitude.	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total for one year.
Boston, Mass.*	42.5	0.9	1.20	1.80	3.10	4.61	5.86	6.78	5.49	4.09	2.95	1.68	1.20	39.61
Rochester, N. Y.*	43.0	0.86	0.86	1.67	2.39	3.45	4.38	4.80	4.54	3.54	2.60	1.46	1.21	31.76
Cincinnati, Ohio*	39.0	1.00	1.50	2.50	4.12	5.07	6.21	7.20	7.26	5.63	3.00	1.50	1.00	45.99
Birmingham, Ala.*	33.6	1.50	1.50	2.25	4.45	5.91	7.28	7.36	7.34	6.00	4.00	2.25	1.50	51.34
Great Lakes*	....	.66	0.84	1.03	2.12	2.76	3.67	4.38	3.98	3.21	2.56	1.67	1.08	27.96
Mitchell, Nebr.†	41.0	1.75	1.75	3.00	4.50	6.25	8.05	10.95	9.39	7.44	5.59	4.00	3.00	65.67
Snake River, Idaho†	42.5	2.25	2.50	4.00	7.00	11.21	12.31	15.00	13.50	11.00	8.50	5.75	3.50	96.52
Boise, Idaho†	43.6	2.00	2.75	4.25	6.00	7.90	9.59	10.59	12.16	9.25	5.42	5.52	2.00	77.43
Fallon, Nev.†	39.0	1.75	1.75	2.25	3.25	5.25	7.86	9.86	8.70	5.13	3.35	2.50	2.00	53.65
Hermiston, Ore.†	45.8	1.25	1.25	3.00	7.28	7.89	9.54	12.04	11.07	7.35	3.88	2.00	1.50	68.05
Klamath, Ore.†	42.2	0.50	1.25	3.57	6.64	7.15	6.99	8.01	9.21	6.13	2.50	1.00	0.50	53.45
North Yakima, Wash.†	46.8	1.75	2.50	6.25	7.91	8.36	8.90	10.74	9.41	5.51	3.15	2.00	1.50	67.88
Tahoe Lake, Calif.†	39.2	1.75	1.75	1.75	2.00	3.00	4.25	6.19	7.08	6.22	3.60	2.62	2.00	42.21
Salton Sea, Calif.†	33.4	3.41	5.09	5.95	8.75	10.50	13.00	14.08	12.19	12.03	9.24	5.96	5.25	105.45
Indio, Calif.†	33.4	3.18	5.08	7.50	12.05	15.84	16.11	16.34	13.78	12.87	8.91	5.17	3.00	119.83
Mecca, Calif.†	33.4	2.92	5.00	8.07	10.87	12.72	14.23	15.21	13.22	10.29	8.17	4.13	2.98	107.81
Brawley, Calif.†	33.0	3.05	5.00	8.00	10.74	13.79	13.68	14.14	11.26	10.15	6.99	4.09	2.66	103.55
Mammoth, Calif.†	33.0	4.24	5.67	8.99	12.02	15.52	16.75	18.00	13.78	12.16	9.49	5.26	3.70	125.58
Phoenix, Ariz.†	33.6	4.25	4.40	5.25	7.00	9.50	12.00	12.75	12.50	11.00	8.31	6.56	4.22	97.74
Lee's Ferry, Ariz ‡	....	1.74	3.52	5.87	7.16	11.70	13.70	13.70	11.33	8.85	6.29	3.98	1.91	89.75
Roosevelt Dam, Ariz.‡	....	2.29	3.10	5.83	7.32	10.37	12.67	12.84	10.56	8.60	5.78	3.52	2.13	84.02
Mesa Experiment Station.‡	....	2.78	3.68	5.74	7.78	10.27	11.17	10.55	8.29	6.39	4.73	3.36	2.62	77.36
Wilcox, Ariz.‡	....	3.59	4.73	7.29	10.10	11.24	12.20	10.64	9.03	8.10	6.70	4.67	3.41	91.70
Yuma Evaporation Station, Ariz.‡	....	3.09	3.89	5.72	7.23	8.24	8.91	10.24	9.96	7.85	5.39	3.40	2.62	76.54
Carlsbad, N. Mex.‡	....	4.50	4.50	5.51	7.45	10.12	11.05	12.88	12.00	9.50	7.00	5.75	4.50	94.76
Agriculture College, N. Mex.‡	....	2.87	4.50	7.41	9.37	11.10	11.91	11.15	9.79	8.09	5.93	3.63	2.52	88.27
Elephant Butte, N. Mex.‡	....	2.78	4.49	7.56	10.44	13.48	14.48	12.22	10.83	9.07	7.76	4.32	3.05	100.48
Lake Avalon, N. Mex ‡	....	2.34	3.26	5.49	7.49	7.87	8.70	10.13	9.60	8.59	6.49	3.17	2.42	75.55
Santa Fé Field, N. Mex.‡	....	1.53	2.13	3.96	6.15	8.51	10.14	8.93	8.06	6.55	4.81	2.55	1.93	65.25
Spur, Tex.‡	....	2.79	3.54	4.82	5.48	6.66	8.68	9.83	7.38	5.59	4.48	3.42	2.34	65.01
Hill Ranch, Tex.‡	....	2.47	3.43	5.29	6.01	6.81	7.95	8.97	8.72	6.36	4.96	3.18	2.44	66.59
Beeville, Tex.‡	....	2.69	3.26	4.26	4.79	6.21	7.35	8.14	8.14	5.89	4.72	3.06	2.17	60.68

* "Regulation of the Great Lakes," by John R. Freeman, Past-President, Am. Soc. C. E., 1926.

† *Engineering-News*, 1910.

‡ *Monthly Weather Review*, July, 1927, p. 320.

It is interesting to note that by using the insolation quantity as computed by Method (2) and by using the value of radiation, (B_r), given for any one of four places mentioned in Table 6, the evaporation computed for the fifth place was within 5% of the observed evaporation in each case.

Before taking up the practical applications of Equation (10) it is well to consider Table 10, which contains good evaporation records for stations in the United States. Unfortunately, these records were from tanks which were not of the same size. It is well also to bear in mind that the amount of evaporation from a small tank next to a lake is not the same as that from the larger body of water, but represents simply an index. The data in Table 10 were compiled from records presented in various places, as noted. Equation (10) may be solved by substituting observed values of I from sources such as Table 3 and by determining values of B_r from Equation (8). Table 11 gives a comparison of the observed and computed evaporation near pyrheliometer stations.

TABLE 11.—COMPARISON BETWEEN OBSERVED AND COMPUTED EVAPORATION.

Station.	Insolation observed at pyrheliometer stations, in inches per year, divided by 1.22.	Back radiation, in inches per year, divided by 1.22.	Computed evaporation, in inches per year.	Observed evaporation, in inches per year.
Boston, Mass.	60.0	17.3	42.7	39.11
New York, N. Y.	50.4	17.3	33.1	31.76
Cincinnati, Ohio	66.0	17.3	48.7	49.99
Great Lakes	49.5	17.3	32.2	27.96
Birmingham, Ala.	66.1	15.0	51.1	51.3
Mitchell, Nebr.	73.7	17.0	56.7	65.67
Boise, Idaho	78	17.3	60.7	77.43
Klamath, Ore.	62.7	17.3	45.4	54.40
Lake Tahoe, Calif.	62.7	17.3	45.4	42.2

Evaporation rates have been computed for the stations given in Table 10 with good correspondence. The results of three other computations were as follows: Roger C. Wells, of the U. S. Geological Survey, determined the evaporation of Chesapeake Bay to be 120.9 cm. per year,[13] while the computed amount, using his excellent data, was 124.3. McEwen by one method and Grunsky by another[14] determined the evaporation of Lake Mendota (near Madison, Wis., a pyrheliometer station) for a 5-month period, to be 41.45 and 45.5 cm., respectively, while the computed amount was 39.66. The evaporation of Swiss Alps lakes,[15] from a 90-day period amounted to 198.3 cm., while the computed amount was 193.7.

When the value of insolation is not known from pyrheliometer records it is possible to compute the insolation, I_{ex}, incident upon the exterior of the earth's atmosphere for any latitude, and, later, to convert this into actual Insolation (I) by means of the two coefficients, ρ' and F, the mean transmission coefficient of the atmosphere at a particular place and the clearness factor, respectively. The relation may be expressed by the integral:

---

[13] *Journal*, Washington Academy of Science, October 19, 1928; U. S. Geological Survey, Dept. of the Interior, *Professional Paper 154C*, March 14, 1929.

[14] Technical Series, Scripps Institution of Oceanography, Univ. of California, Vol. 2, No. 6, September, 1929, pp. 177–306.

[15] *Monthly Weather Review*, August, 1925, p. 355.

$$I_{ex} = 2\,K \int_{0}^{t} = \cos \psi \, d\, t \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Integrating, after writing for $\cos \psi$ its equivalent, $\sin \lambda \sin d + \cos \lambda \cos d \cos w\, t$, Equation (11) becomes:

$$I_{ex} = (2\,K\,t \sin \lambda \sin d) + \left(\frac{2\,K}{w} \cos \lambda \cos d \sin w\, t\right) \ldots\ldots (12)$$

in which, $\lambda =$ the latitude of the station; $d =$ the sun's declination for a certain day of the year; and $w = \dfrac{2\,\pi}{24 \times 60} =$ the angular velocity of the earth about its axis.

TABLE 12.—EVAPORATION AS DETERMINED BY COMPUTING THE INSOLATION RECEIVED ON THE EXTERIOR OF THE EARTH'S ATMOSPHERE.

Values of λ, in degrees north latitude.	VALUES OF I_{ex}, IN CALORIES PER SQUARE CENTIMETER PER DAY.											
	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.
52	189	305	512	743	916	993	953	818	527	387	226	161
49	229	347	548	765	929	1 003	965	837	562	425	270	198
48	241	359	557	770	932	1 000	964	838	579	441	283	211
45	286	404	596	796	945	1 008	976	861	611	478	323	255
42	322	440	623	809	948	1 004	974	870	638	510	361	291
41	336	450	634	815	951	1 002	976	871	647	521	376	291
37	392	505	678	844	963	1 032	993	896	696	572	433	367
33	449	556	717	864	963	1 003	985	908	731	619	489	424
30	487	585	785	871	960	992	977	911	799	646	519	461
29	491	600	746	875	956	990	974	913	807	651	533	476
20	618	702	714	904	947	958	851	925	856	746	642	595

Values of λ, in degrees north latitude	VALUES OF EVAPORATION, E, IN INCHES OF EVAPORATION FROM A WATER SURFACE CORRESPONDING TO THE ABOVE VALUES OF I_{ex}.											
52	3.9	5.7	10.6	14.9	18.9	19.9	19.7	16.9	10.5	8.0	4.5	3.4
49	4.7	6.5	11.3	15.3	19.2	20.1	19.9	17.3	11.2	8.8	5.4	4.1
48	5.0	6.7	11.5	15.4	19.3	20.1	19.9	17.3	11.6	9.1	5.7	4.4
45	5.9	7.5	12.3	15.9	19.5	20.2	20.2	17.8	12.2	9.9	6.5	5.3
42	6.7	8.2	12.9	16.2	19.5	20.1	20.2	18.0	12.8	10.5	17.2	6.0
41	6.9	8.4	13.1	16.3	19.6	20.1	20.2	18.0	12.9	10.8	7.5	6.3
37	8.1	9.4	14.0	16.9	19.9	20.6	20.5	18.5	13.9	11.8	8.7	7.6
33	9.3	10.4	14.8	17.3	19.9	20.1	20.4	18.8	14.6	12.8	9.8	8.8
30	10.1	10.9	15.2	17.4	19.8	19.8	20.2	18.9	16.0	13.3	10.4	9.5
29	10.3	11.2	15.4	17.5	19.8	19.8	20.1	18.9	16.1	13.4	10.7	9.8
20	12.8	13.1	14.8	18,1	19.6	19.2	19.6	19.1	17.1	15.4	12.8	12.3

The time, t, is the number of minutes from noon to sunset, and the value of Abbot's solar constant, K, is 1.93 calories per sq. cm. per min. Values of

TABLE 13.—THE RATIO OF OBSERVED INSOLATION AND THE COMPUTED INSOLATION STRIKING A HORIZONTAL SURFACE ON THE EXTERIOR OF THE EARTH'S ATMOSPHERE FOR ALL STATIONS WHERE INSOLATION RECORDS HAVE BEEN KEPT.

Station.	Years observed.	Values of the Ratio, $\frac{I}{I_{ex}}$.													
		January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Average monthly.	Deviation from the mean.
New York, N. Y.	1924–1929	0.34	0.42	0.40	0.45	0.39	0.36	0.40	0.31	0.38	0.38	0.30	0.31	0.37	0.038
Washington, D. C.	1924–1929	0.47	0.42	0.50	0.51	0.48	0.48	0.46	0.47	0.51	0.51	0.46	0.44	0.48	0.026
Chicago, Ill.	1924–1929	0.31	0.28	0.35	0.42	0.45	0.33	0.38	0.43	0.49	0.33	0.24	0.27	0.36	0.069
Madison, Wis.	1924–1929	0.55	0.46	0.50	0.53	0.48	0.52	0.47	0.49	0.49	0.42	0.39	0.45	0.38	0.034
Lincoln, Nebr.	1924–1929	0.64	0.53	0.58	0.57	0.49	0.56	0.56	0.55	0.57	0.56	0.54	0.57	0.56	0.023
Twin Falls, Idaho	1927–1928	0.53	0.48	0.39	0.44	0.40	0.60	0.70	0.80	0.82	0.77	0.51	0.54	0.58	0.13
Vancouver, B. C., Canada	1924	0.41	0.43	0.37	0.36	0.39	0.35	0.35	0.38	0.35	0.38	0.38	0.35	0.38	0.020
Victoria. B. C., Canada	1924	0.38	0.40	0.44	0.42	0.43	0.40	0.41	0.41	0.41	0.38	0.36	0.35	0.41	0.024
St. Johns, N. B., Canada	1924	0.46	0.52	0.46	0.42	0.42	0.42	0.44	0.45	0.46	0.41	0.43	0.39	0.44	0.025
Boat off Boston, Mass.	1924	0.42	0.47	0.50	0.47	0.47	0.47	0.44	0.44	0.47	0.50	0.46	0.43	0.46	0.020
Near Eureka, Calif.	1924	0.45	0.52	0.48	0.51	0.51	0.52	0.46	0.45	0.47	0.49	0.46	0.46	0.48	0.024
Boat off Florida	1924	0.43	0.40	0.48	0.47	0.47	0.44	0.44	0.44	0.42	0.42	0.49	0.43	0.45	0.022
Lower California, Mexico	1924	0.62	0.59	0.55	0.54	0.49	0.48	0.46	0.49	0.54	0.55	0.61	0.62	0.55	0.045
Santiago, Cuba	1924	0.54	0.58	0.54	0.52	0.48	0.52	0.49	0.49	0.51	0.52	0.52	0.52	0.52	0.018
Pasadena, Calif.	1926–1927	0.70	0.70	0.61	0.63	0.59	0.51	0.59	0.63	0.63	0.65	0.76	0.75	0.65	0.056
Fort Collins, Colo.	1926	......	......	......	......	......	......	......	......	0.57	......	......	......	......	......
San Francisco, Calif.	1927	......	......	......	......	......	......	......	0.65	......	......	......	......	......	......
San Diego, Calif.	1927	......	......	......	......	......	......	0.69	......	......	......	......	......	......	......
La Jolla, Calif.	1928–1929	......	......	......	......	......	......	0.56	......	......	......	......	......	......	......

declination, d, vary from $+ 23°$ on June 21 to $- 23°$ on December 21 of each year. The results of applying these values in Equation (12) to obtain quantitatively the insolation striking the exterior of the earth's atmosphere are given in Table 3 in "Computed Insolation." Table 12 gives additional values of exterior insolation for eleven different latitudes extending from Canada to Mexico, together with the equivalent, in inches of evaporation. These values of evaporation would represent the physical maximum because they assume a constant back radiation, no clouds, and a transmission coefficient of 1.00.

CLEARNESS FACTORS

At the beginning of this paper the writer stated that there is need for a unit of cloudiness and for records of cloudiness, kept in a uniform manner. At present, this factor may be expressed as a clearness factor to be computed by Kimball's formula:[16]

$$F = \frac{Q}{Q_s} = 0.22 + 0.78\,(1.0 - C') \ldots\ldots\ldots\ldots\ldots (13)$$

in which, F is a clearness factor; Q is the observed radiant energy that passes through local clouds; Q_s, the observed radiant energy that passes through a clear sky; and C' the percentage of cloudiness that occurred during a certain month as given in *Monthly Weather Review* reports.

This value is useful in converting the insolation, I_{ex}, on the exterior of the earth's atmosphere to surface insolation, I, and to evaporation. In algebraic terms:

$$I = I_{ex}\,\rho'\,F \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Equation (14) demonstrates that the combined effect of relative cloudiness and the atmospheric transmission coefficient may be expressed as the ratio, $\frac{I}{I_{ex}}$. This is an important reason for accumulating insolation data. Table 13 includes values of this ratio.

CONCLUSIONS

It is possible to determine the value of insolation by three distinct methods, and the results obtained by these different methods check experimentally within 5 per cent. The six Government stations that record insolation are mostly in the eastern part of the United States and more stations in the Western States are highly desirable. Where it is not possible to get a ratio of evaporation to insolation from pyrheliometer records, it is necessary to compute the insolation on the exterior of the earth's atmosphere and correct this quantity by a transmission coefficient for the earth's atmosphere and by a clearness factor (see Equation (14)). The evaporation formula (Equation (9)) given in terms of insolation and radiation checks experimentally with observed evaporation in California, and when applied to bodies of water outside California it gives satisfactory results.

The author is indebted to Franklin Thomas, M. Am. Soc. C. E., for encouragement that led to the preparation of this paper.

---

[16] *Monthly Weather Review*, April, 1927, p. 156.

DISCUSSION

H. H. KIMBALL,[17] ESQ. (by letter).—That a close relation exists between insolation and evaporation is well known. Thus, in a paper entitled "Applications of Heat Radiation Measurements to the Problems of the Evaporation from Lakes and the Heat Convection at Their Surfaces",[18] A. Angström assumes a balance between (1) the insolation absorbed at a water surface; (2) the heat loss from the surface through radiation, convection, and conduction; (3) the heat energy expended in raising the temperature of the water; and (4) in the process of evaporation at its surface. It is evident that if Components (1), (2), and (3), of this balanced system, can be measured or computed, Component (4) will be known.

However, when, at a later date, in a paper entitled "On Radiation and Climate",[19] Angström undertook to account for the expenditure of the radiant energy received from the sun, he computed, from evaporation measurements and the latent heat of evaporation, the energy expended in evaporating moisture from various surfaces, including water surfaces.

In his paper, the author goes back to Angström's earlier method, shows how the values of various terms may be computed, and states that, if the quantity of insolation received at the water surface is known, the rate of evaporation may be computed; or, *vice versa,* if the rate of evaporation is known, the insolation may be computed.

In Table 11, comparisons are given between observed and computed amounts of evaporation, the computed amounts being based on "insolation observed at pyrheliometer stations."

Pyrheliometric readings are available for only a few of the stations named in the table. Therefore, the values given for insolation cannot be considered of a high order of accuracy.

The author states that,

> "When the value of insolation is not known from pyrheliometer records it is possible to compute the insolation, I_{ex}, incident upon the exterior of the earth's atmosphere at any latitude, and, later, to convert this into actual Insolation I by means of two coefficients ρ' and F, the mean transmission coefficient of the atmosphere at a particular place and the clearness factor, respectively."

This is true if the atmospheric conditions are known, as has been shown by the writer in a recent paper entitled "Measurements of Solar Radiation Intensity and Determinations of Its Depletion by the Atmosphere"[20]; but it will by no means do to assume an annual mean value for ρ', which is dependent especially upon the water vapor and dust content of the atmosphere. Both these factors are subject to marked seasonal or annual variations, especially near sea level. (See, for example, values of ρ', or a_{0-1}, for Washington, D. C., determined for zenithal sun, $\psi = 0°$.)

---

[17] In Chg., Solar Radiation Investigations, U. S. Weather Bureau, Washington, D. C.

[18] *Geografiska Annaler,* 1920, H. 3.

[19] *Loc. cit.,* 1925, H. 1, och. 2.

[20] *Monthly Weather Review,* Vol. 58, February, 1930, pp. 43–52.

C. G. ABBOT,[21] ESQ. (by letter).—While no doubt valuable in engineering estimates the use of the coefficient of atmospheric transmission as a single valued quantity is not, of course, exact, for it varies with the wave length, with the station, with the month, and from day to day in a very wide range. Therefore, the results reached by treating it as a single valued constant can be no more than rough average approximations.

D. P. KRYNINE,[22] M. AM. SOC. C. E. (by letter).—At first glance this paper seems to be very theoretical. Nevertheless, it must be considered as having a particular interest both for research men and practising engineers working in those fields of engineering connected with soils, water, and evaporation. The basic idea of the proposed methods seems to be correct, but for practical application, corrections corresponding to points of different longitude, due to local circumstances, must be studied further. Climatology and meteorology as applied to Civil Engineering is a broad field which has scarcely been touched. The writer has in mind one other work dealing with this subject from an engineering point of view.[23]

Determination of the Zenith Angle.—The author suggests determining the value of the zenith angle, ψ, by methods described by Bowditch. The writer, however, recommends the equation:

$$\cos \psi = \sin \lambda \sin d + \cos \lambda \cos d \cos \omega t \ldots\ldots\ldots\ldots (15)$$

From this formula,[24] at a given time, t, the angle, ψ, may easily be found. The formula becomes very simple, if the observation is performed at noon. In this case the hour angle may be assumed equal to zero, and $\cos \psi = \sin \lambda \sin d + \cos \lambda \cos d$, or,

$$\psi = \lambda - d \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The methods proposed by Mr. Bowditch are based on Equation (15) and require the use of "American Ephemeris and Nautical Almanac."

Determination of the Insolation Incident Upon the Exterior of the Earth's Atmosphere.—In Equation (15) the declination, d, is not constant during a given day, but changes gradually, being thus a function of time. Therefore, strictly speaking, the integration, Equation (12), must be considered as approximate. However, the first derivative of the declination with respect to time is small (less than 1′ of arc per hour), and introducing a mean value of d, as suggested by Mr. Richardson, does not involve a serious error, especially since the coefficients, ρ' and F, probably, are sufficiently far from being accurate.

The writer regrets that only a few data are given in the paper with respect to the mean transmission coefficient, ρ', of the atmosphere at a particular place.

Research to Determine Values of the Coefficient, ρ; Determination of the Values of A.—The method of determining the coefficient, ρ, from Equation

---

[21] Secy., Smithsonian Inst., Washington, D. C.

[22] Research Associate in Soil Mechanics, Dept. of Civ. Eng., Yale Univ., New Haven, Conn.

[23] "The Influence of Climate on the Building, Maintenance, and Use of Roads in the United States," by F. H. Eno, M. Am. Soc. C. E., *Proceedings*, Ninth Annual Meeting, Highway Research Board, December, 1929, p. 211.

[24] The letter, d, corresponds to the declination of the sun; λ is the latitude; and ω, the angular velocity of the earth about its axis.

(2) as proposed by the author, is both simple and efficient. Actually, after using the pyrheliometer for determining ρ, the investigator does not need that instrument any more and may complete the remainder of the computations referring to evaporation without leaving his desk.

However, the difficulty with the method proposed by the author is due to the necessity of applying the coefficient, A, which represents the ratio of solar radiation to total radiation. The writer understands from Table 5 that the values of A are known only for a few localities.

For practical engineering purposes probably a rough estimation of the coefficient, A, is all that is necessary. Suppose the experiment is begun by taking the reading of a pyrheliometer corresponding to a zenith angle as close to zero as possible in given circumstances. The notation proposed is given in Table 14.

TABLE 14.—NOTATION FOR DETERMINING APPROXIMATE VALUES OF A_n.

Item.		Initial reading on the pyrheliometer.	Any subsequent reading.	Imaginary reading (zenith angle being equal to 0).
(1)	Zenith angle......................	ψ	ψ_n	ψ_0 (= 0)
(2)	Secant of the zenith angle..........	u	U	u_0 (= 1)
(3)	Value of the pyrheliometer reading, $\frac{dI}{dt}$....................	p	p_n	p_0
(4)	Correction coefficient.............	A	A_n	A_0

By substituting these symbols, Equation (3) becomes:

$$p = \frac{K}{A} \times \frac{\rho^u}{u} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(17)$$

Suppose, now, that another pyrheliometer reading is taken at such a time that the secant of the new zenith angle is mu, in which, m is an arbitrary value. In this case, Equation (17) would be:

$$p_n = \frac{K}{A_n} \times \frac{\rho^{mu}}{mu} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(18)$$

Raise both sides of Equation (17) to the m power:

$$p^m = \left(\frac{K}{A}\right)^m \frac{\rho^{mu}}{u^m} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(19)$$

and solve simultaneously with Equation (18), thus:

$$\left(\frac{A\,p}{K}\right)^m u^m = \frac{A_n\,p_n}{K} \times mu \ldots\ldots\ldots\ldots\ldots\ldots(20)$$

Next, assume that the first reading on the pyrheliometer was made when the zenith angle was equal to zero. It may be possible that in the given locality such an angle may never be observed; but, nevertheless, Equation

(20) may be applied to such an imaginary case, by substituting the values: $A = A_0$; $p = p_0$; $u = u_0$; and $m = U$. Then, Equation (20) becomes:

$$A_n = \left(\frac{A_0\, p_0}{K}\right)^U \; \frac{p_n\, U}{K} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Equation (21) includes three unknowns, A_n, A_0, and p_0. In Table 5, $A_0 = 0.87$ for $\psi = 0$ at all latitudes observed. Suppose the same condition occurs in the present case and that an approximate value of p_0 is:

$$p_0 = u\, p \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Then a formula may be obtained by which to make a quick determination of the coefficient, A_n:

$$A_n = \left(\frac{0.87\, u\, p}{K}\right)^U : \frac{p_n\, U}{K} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

in which, K is the solar constant.

An examination of the function, A_n, and of its derivative with respect to U, will give an idea of the shape of the curve representing A_n.

Isolines of f(u).—Let Equation (17) be written in the form:

$$p = \frac{f(u)}{A}$$

If the pyrheliometer's readings for a series of zenith angles are taken, the values of A may be easily computed. (See Equation (23).) Thus, in a given locality, all the values of $f(u)$ are also known. It is possible to represent $f(u)$ by drawing isolines for different zenith angles, within a certain territory. Then it would be possible to determine insolation and, consequently, the evaporation, at any day and at any locality within the area so plotted. The investigator must use the "Nautical Almanac", in order to determine the minimum zenith angle corresponding to the given day in a given locality and the times at which the zenith angles shown on the isoline maps take place. Interpolation may also be needed. This idea, naturally, is rough and is subject to improvement. The assumption is also made that $f(u)$ is not subject to considerable seasonal variations.

The accurate determination of the coefficient, ρ, as such, is not essential for a civil engineer. It is possible, however, to estimate the value of ρ, even with a certain degree of accuracy, from the approximate pyrheliometer reading for the zero zenith angle:

$$\rho = \frac{p_0}{K} = \frac{p\, u}{K} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

Method of Measuring Evaporation.—The author suggests computing evaporation, E, by means of Equation (10).

The value of B_r, in this equation is, in turn, given by a rather complicated formula, (Equation (8)) and varies between 117 calories per sq. cm. per day to 148 calories per sq. cm. per day (Table 6), the average value (weighted) being 131.4. Considering this average value as a 100%, the lower limit of B_r would be equal to 89% and the upper limit, to 113%, the deviation from the average being thus 11 to 13 per cent. Considering the fact that the back radiation—as a factor that prevents evaporation—may be evaluated as 23 to 35%

(see Table 11), its final influence on the result is between $2\frac{1}{2}$ and $4\frac{1}{2}\%$, the average being about $3\frac{1}{2}$ per cent. Since the result of the computation may be only approximate, it seems to be advisable to change Equation (10) somewhat as follows:

$$E = \frac{I - 131.4}{c} \quad \text{(25)}$$

or, in general:

$$E = \frac{I}{c} - c_1 \quad \text{(26)}$$

in which, c and c_1 are constants.

Clearness Factors.—Personally, the writer is very much interested in the problem of determining clearness factors, since it may have some application not only in irrigation, but also to highway engineering. Certain road treatments involving the use of liquids ought to be different on sunny and shaded sections of a road. For instance, forest sections seem to need surface treatment different from field sections. The writer was led to these conclusions during an examination of the experimental road at Branford, for the Connecticut State Highway Department.

Obviously, the "heavy machinery" of zenith-angle determinations and integral calculus is not necessary in improving modest country roads, but there is a need for a rough field method of determining clearness factors to be used in such work.

R. C. WELLS,[25] ESQ. (by letter).[26]—This paper is of special interest because it approaches the subject of evaporation from a new point of view. It covers rather a wide range of geo-physical and meteorological measurements, and too much space would be required to discuss these in detail. Some of the measurements can be made with considerable accuracy, but others depend on natural conditions, such as clearness of the sky, the transmission coefficient of the atmosphere, etc., and, in view of the fact that some observations are incorporated in the author's calculations in the form of averages over limited periods, the agreement shown between calculated and observed evaporation is surprisingly good.

A reference[27] is made to the work of the writer concerning evaporation from Chesapeake Bay, from which it would appear that this was actually measured. This was not the case; the figures quoted by the author as the determined evaporation refer to the rainfall. In a case like that of Chesapeake Bay no check on the calculated evaporation is possible. On the other hand, when comparisons are made between observed and calculated evaporation for pans, the observed conditions are not generally what they would be over large bodies of water, so that it is not clear just what figures should be used in calculating the evaporation for the larger bodies of water.

---

[25] Chf. Chemist, U. S. Geological Survey, Washington, D. C.

[26] Published with permission of the Director of the U. S. Geological Survey.

[27] *Journal*, Washington Academy of Science, October 19, 1928; U. S. Geological Survey, Dept. of the Interior, *Professional Paper 154C*, March 4, 1929.

The writer would be interested to have the notation explained somewhat more fully for H, P_w, T_a, and T_w, especially the exact distances, or approximate distances, meant by the words "above the water" and "over the water". He agrees fully with the author concerning the great scarcity of reliable observations on evaporation, and is glad to see a definite number of ascertainable features listed for further measurement.

It is noted that in Table 11 the observed evaporation is lower than the lowest calculated, as well as higher than the highest calculated. Is this due to the use of a constant value for R, which is the ratio of convection to evaporation? Convection must express climatic conditions to a considerable extent and records for convection for different localities would be of interest. In fact, they are probably similar to those for evaporation; yet the figures in Table 7 show that convection is variable from hour to hour and from day to day, and R is there much more variable than E, the evaporation.

Another matter that perhaps should be discussed further before this problem is settled, is the bearing of the fact that insolation is practically zero at night whereas evaporation continues, and, in view of this fact, whether a variable that governs only when the sun shines furnishes as direct and as sound a basis for computing evaporation as some other that controls under all conditions.

The author has made a distinct contribution in tracing the heat gains and losses that may affect evaporation.

N. W. CUMMINGS,[28] ESQ. (by letter).—It may be some time before the true value of Mr. Richardson's paper is fully appreciated, because economic pressure has not yet compelled the abandonment of unnecessary complications in estimating evaporation from lakes and reservoirs. For example, there are now various "standard" types of pan differing with respect to dimensions and method of installation. Those investigators who are accustomed to any one type are desirous of seeing that type perpetuated and its use extended so as to make readings at different times and places "comparable" with the ones with which they are already acquainted.

The manner in which any type of pan is to be made comparable to any lake is a mystery which many writers avoid mentioning, although it is difficult to see what benefit can be derived from making various pans comparable to each other unless they are comparable to lakes.

The late R. B. Sleight, Assoc. M. Am. Soc. C. E., found[29] that the ratio of the depth of evaporation from a pan 1 ft. in diameter is 155, while that from a pan 2 ft. in diameter is 129, and that from one 3.39 ft. in diameter is 121, on the assumption that the value for a pan 12 ft. in diameter is 100. The complete explanation of this variation will not be attempted here, but heat leakage through the walls undoubtedly contributes largely to it, as is shown by the fact that the divergence vanishes almost entirely when thermally insulated vessels are used. This is proved by the results of observations made at the Scripps Institution of Oceanography, University of California, from

---

[28] Associate in Physics, The Scripps Inst. of Oceanography, Univ. of California, La Jolla, Calif.

[29] *Journal of Agricultural Research*, Vol. 10, No. 5, July 30, 1917, pp. 209–261.

September 11 to December 23, 1929. During this interval of time many varieties of weather were experienced.

A thermally insulated pan, 2 ft. square and 8 in. deep, and a 1-gal. vacuum-insulated jar were run side by side. The diameter of the mouth of the jar was 6 in.; yet the depth of loss from the two vessels differed by only 1.5 per cent. Furthermore, the correlation coefficient between accumulated departures of evaporation and accumulated departures of insolation was 0.91 ± 0.01; while the corresponding coefficient for evaporation and wind was 0.01 ± 0.07.

These experiments show that important simplifications result from insulating the pan thermally and basing evaporation calculations on energy considerations, because difficulties concerning dimensions and manner of exposure are either eliminated or reduced materially.

Burt Richardson, Esq. (by letter).—Much interest has been shown in this study of the evaporation from lakes, and the writer was gratified to receive inquiries regarding the possibility of computing the evaporation quantitatively from a knowledge of insolation. As might be expected the Western States showed the most interest in the subject. For instance, the Forestry Department of the University of California made inquiries; graduate students of the University of Southern California repeated the experiment described in the paper; the United States Naval Air Station, at San Diego, Calif., worked on a fog-prediction method and, therefore, showed its interest in evaporation and insolation data; the Scripps Institution of Oceanography of the University of California, at La Jolla, developed a plan for rainfall prediction for Southern California, in which evaporation and insolation are two important quantities; while the motion picture industry and the Stanford University Marine Biological Station welcomed the insolation and atmospheric absorption data.

The evaporation from a heat-insulated tank can always be computed with excellent results when the experiment is made with a pyrheliometer, sensitive instruments, and good atmospheric data such as those furnished by the Mount Wilson Observatory; but when the quantity of evaporation is from a lake, say, in Beesville, Tex., where no pyrheliometer station is located, where the absorption coefficient has never been computed, and where the observed value of evaporation has been measured by some caretaker of the lake—then the results must of necessity be rough average approximations, as suggested by Dr. Abbot. However, limiting values of each term used in the evaporation formula are known, and their use sets definite limits to the quantity of evaporation. In any event, these evaporation estimates should be more accurate than applying to a lake a method which is based on a knowledge of the rainfall and an estimate of the quantity of inflowing and outflowing water with a correction for seepage.

The mathematical "short-cuts" suggested by Professor Krynine are quite welcome. According to information received from the Mount Wilson Observatory, the values of the two terms, A and ρ, used in this paper are not known for places in the United States other than those mentioned. The method of making observations of temperatures of water surface, wet-bulb, and dry-bulb, which are required in computing the terms, H, P_w, T_a, and T_w, is rather fully described in the two references to the *Physical Review*.

No difficulty presents itself in applying the evaporation formula to a lake during the night when, of course, the insolation is equal to zero. At that time, if the water is warmer than the air, the heat that goes into evaporation, as well as the radiation, conduction, and convection, is taken, calorie for calorie, from the quantity of sensible heat stored in the body of the water.

Professor Cummings[30] has recently done some experimental work which shows clearly the rate of evaporation from tanks of various sizes. He has also shown the relation of evaporation to humidity and convection.

Since presenting this paper the writer has computed the evaporation from most of the lakes of the United States where evaporation had been previously observed, and the results obtained by comparing the computed and observed quantities have been satisfactory. The best results obtained were in cases where the lakes were near pyrheliometer stations.

The writer is convinced of the importance of evaporation studies and will continue to urge those who are interested in the storage of water to gather and publish data regarding observed evaporation and temperatures, which have been made in a uniform manner. He also will continue to urge those institutions that are financially able to do so to install Weather Bureau pyrheliometers; and he will attempt to interest astronomers and meteorologists in studying the absorption coefficient of the earth's atmosphere, in order that, in the future, a better knowledge of evaporation may be had.

---

[30] "Relation between Evaporation and Humidity", by N. W. Cummings, *Bulletin No. 68*, National Research Council (1929), pp. 47–56; also "Alignment Diagram for Ratio of Convection to Evaporation and Limits of Ratio of Convection to Evaporation," by N. W. Cummings, *Monthly Weather Review*, Vol. 58 (1930), pp. 142–146.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

TRANSACTIONS

Paper No. 1783

HIGHWAYS AS ELEMENTS OF TRANSPORTATION[1]

By Fred Lavis,[2] M. Am. Soc. C. E.

With Discussion by Messrs. W. W. Crosby, Walter Loring Webb, Jay Downer, S. Johannesson, D. P. Krynine, Harold M. Lewis, Donald M. Baker, Edwin F. Wendt, H. W. Hudson, and Fred Lavis.

Synopsis

The paper shows that, for handling large volumes of traffic through congested areas, highways should be designed for the main purpose of transportation rather than as adjuncts of the abutting properties. It is pointed out that where congestion of existing city streets interferes with the reasonably free passage of vehicles and pedestrians, highways of the type described are necessary, not only to expedite through traffic, but also to restore such streets to their normal uses.

The author includes a brief description of the through trunk highway recently built by the State of New Jersey through Jersey City and Newark; discusses the factors which influenced its location; and develops a general economic theory which governs the relation between cost of construction and the operating costs of vehicles using highways of this character. This theory is based on the principles of Wellington's "Economic Theory of Railway Location."

Historical Development

Up to within very recent times, highways in large part have been developments of the old trails made by man before the eras of wheeled vehicles, or they have been laid out as means of reaching real property. Their design and construction as elements of transportation are very new and this is required of course, by the intensive development and use of motor vehicles.

---

[1] Published in August, 1930, *Proceedings*.

[2] Pres., International Rys. of Central America, New York, N. Y.

Older Highways.—Economic factors developed by the use of hard surfaces for highways were probably first recognized by Macadam and others at the end of the Eighteenth and the beginning of the Nineteenth Century. Experiments to determine the tractive effort required for haulage over various types of road surfaces were made by several investigators between 1840 and 1850.

In spite of the fact, however, that the economic relation between road surfaces, and the tractive effort required to haul loads, was recognized nearly a century ago, by a few competent engineers—men who recognized the principle that true engineering and economics are indissolubly linked together—there has not been, at least until recently, any attempt to actually evaluate the effect of the different elements of the location and types of construction on the cost of operation of the vehicles using the highways.

Motor Vehicles and Modern Highways.—The introduction of the motor vehicle has had a very notable effect on the construction of highway pavements, and has resulted also in certain improvements in alignment, rates of gradient, rise and fall, etc. Expenditures for these purposes have been enormous and are continuing at a rate which is almost staggering. Changes in the locations of existing highways, however, are quite definitely limited by long established conditions; such roads are an integral part of individual, as well as municipal and State, property rights; and any contemplated change must take these rights into consideration.

There has developed, however, in recent years a demand for a new type of road—a road more nearly analogous to the railroad; laid out on an entirely new right of way and designed and built primarily to meet a demand for transportation. Such roads must necessarily be designed from the point of view of their users rather than that of owners of abutting property.

The design and construction of this more modern road are based primarily on the needs of a large volume of intensely mobile traffic which desires to move from one place to another with the least resistance compatible with the physical conditions of the territory through which it must pass. It is a development of modern transportation which must be recognized and provided for.

Transportation a Factor.—The last century has shown the ever-increasing necessity and value of transportation as a factor in the development of commerce and as perhaps the most important and dominant factor in modern civilization.

The railway was and probably still is the most important element in this modern transportation development, other factors being steamships, motor vehicles, air craft, the telegraph, the telephone, and the radio. Modern business requires that railway traffic should not be delayed by stops at grade crossings, and the modern efficient railway is designed with alignment, rates of gradient, rise and fall, multiple tracks, and every adjunct necessary to permit the free rapid movement of its traffic.

These same conditions apply in degree to the type of highway herein described. The volume of railroad business handled has permitted large expenditures for improvements in the road-bed and for the removal of resistances and delays to the passage of trains, so that freight and passengers are not only moved in quantity at high rates of speed, but also at low cost. Highways must be developed to permit the same safe uninterrupted flow of heavy traffic as now obtains on a modern trunk-line railway, and the cost of operating these motor vehicles becomes an economic factor of importance.

These results on the railways have been achieved by a careful consideration of the economic values involved, the relation between location, costs of construction, and costs of operation. A similar era is impending in connection with the construction of highways. The following discussion follows the principle of the "Economic Theory of Railway Location," developed by the late Arthur M. Wellington, M. Am. Soc. C. E.

Need of Special Highways.—It is not only necessary to provide smooth pavements as a matter of comfort and convenience, but it is more than ever necessary where the traffic demand is large, to provide for the free uninterrupted flow of the through traffic without interference with local uses.

In open country the ordinary types of modern highways can and do carry this traffic with reasonable efficiency and dispatch, but in the vicinity of large cities, and in these cities themselves, special provision for through traffic which there reaches large proportions, must be made so that the ordinary city streets may serve local needs and abutting property.

It is with the design of these special highways that this paper is principally concerned and its object is to emphasize the need of making such design and considering such highways on the basis of their efficiency as elements of transportation. These are trunk highways, located on their own rights of way and designed primarily for the economic movement of a large volume of traffic through a given area or between given points.

A MODERN HIGHWAY

Route 25, New Jersey.—The nature of the problem and suggested methods of solution can best be indicated perhaps by a description of the design and construction of the arterial highway between the Hudson River and Elizabeth, known as Route 25 of the New Jersey State Highways. This is also the eastern end of the transcontinental Lincoln Highway. It has a roadway 50 ft. wide, sufficient for five lanes of traffic, maximum gradients of 3.5%, and curves with a minimum radius of 1 000 ft.

At its eastern end (Fig. 1) it connects with the Holland Vehicular Tunnel under the Hudson River. Thence, it passes through Jersey City and thus over the trap dike which divides the Hudson River from the valley of the Lower Hackensack and Passaic Rivers (both being navigable streams), crosses this valley in the lower part of the Town of Kearny, then by-passes the City of Newark, skirting the edge of the uplands and meadows, and, finally, crosses the meadows to the City of Elizabeth, beyond which it connects with the main State highway routes leading toward the South.

The first 8 miles of this route are without crossings of other highways at grade, thus providing for the free, uninterrupted flow of traffic through the highly congested areas of Jersey City and Newark. Provision is made, of course, for connections with important highways by ramps, thus avoiding all cross-traffic currents. This 8-mile section was estimated to cost $25 000 000, and, in connection with its location and design, a careful study was made of the governing economic factors.

General Considerations Affecting Design.—The so-called Metropolitan Area of Northern New Jersey comprises the Cities of Bayonne, Jersey City, Elizabeth, Newark, the Oranges, Montclair, Paterson, and Passaic, and other smaller communities. From a traffic point of view it constitutes a continuous, densely populated, barrier between the City of New York and the South and West of the United States. It is also in itself both a source and a destination of a large volume of traffic.

The Western States may be reached from New York City by traveling northward along the Hudson River Valley to Albany, N. Y., and then striking west, but the natural route is to cross the Hudson from New York City to New Jersey and continue westerly. Such traffic must then necessarily pass through this congested zone, as must, of course, all traffic destined to the Jersey Coast resorts, to Trenton, Philadelphia, Baltimore, Washington, and the South.

The construction of the Holland Vehicular Tunnel concentrated a large part of this trans-fluvial and transcontinental traffic at the lower end of New York and at the most congested area of Jersey City. West of the Hackensack and Passaic River Valleys the traffic again had to find its way by many twists and turns through the overburdened streets of the Cities of Newark and Elizabeth.

The primary cause for consideration of this highway was the need of relief for these overburdened streets in Jersey City, Newark, and Elizabeth, traffic on which had become so congested that neither citizens who desired to shop, shopkeepers who desired to sell, or manufacturers who desired to move materials, could do business except at great expense and delay.

Traffic counts at fourteen points in the City of Newark showed increases for the decade, 1912 to 1922, ranging from 64% to 1 183%, the average being 340 per cent. The increase in traffic on the main State highway at Rahway just south of Elizabeth between 1921 and 1926 was from about 12 000 to 44 000 vehicles for a 24-hour period, that is, nearly fourfold in five years. Other counts indicated similar increases throughout the region.

Most of Northern New Jersey is not only a manufacturing district of great importance, but one so conveniently accessible to the Port and City of New York and to railroad and highway transportation, to a great labor market, and to financial centers, that it is destined to continue to grow, and probably grow very rapidly, in importance, both as a manufacturing, retail business, and residential district.

This highway, therefore, has two important purposes: It will permit the through traffic to cross this congested area, leaving existing city streets and highways free for their normal local uses; and it will permit easy, rapid access between various points in this area, and between all these points and New York City. Further, the great airport built by the City of Newark, although separated from Broadway and the down-town business district of New York City by 10 miles of congested and densely built-up territory, can be reached over Route 25 within 20 to 30 min.

Transportation to Be Provided

Amount of Traffic.—In developing the economic factors to determine values of the physical characteristics of this highway, curves were platted showing the actual existing traffic, its increase during the previous few years, and its probable future increase. These estimates not only were based on counts of traffic, motor vehicle registrations, etc., but they were checked by traffic statistics of other carriers, such as the ferries, railways, and the Holland Tunnel, by charts of the growth of population and use of motor vehicles, by similar statistics of other areas, etc., and by the peculiarities of this particular district.

The application of these data to the future is necessarily largely a matter of judgment, but, reviewing all the information assembled, it seemed entirely probable that the highway would be used to capacity very shortly after its completion. It then remained to determine its capacity, or the number of vehicles which would use it, in order to obtain a basis for the determination of the economic values.

Highway Capacity.—The highway is built with a 50-ft. roadway and is expected to carry four lanes of continuously moving traffic with one 10-ft. lane to spare.

Observations taken (which it is believed check with studies by others) indicated that with freely moving traffic the largest number of vehicles pass a given point when moving at the rate of about 15 to 20 miles per hour. On this assumption four lanes would give a maximum capacity of 5 455 vehicles per hour. It is well known, of course, that traffic not only varies throughout the hours of the day, but also on different days of the week and at different seasons.

The number of vehicles which can use any given highway at times of maximum demand also depends on the proportion of trucks and buses or other types of heavy vehicles to lighter or non-commercial types. (It is not expected that horse-drawn vehicles will use this highway.)

On Route 25, it was necessary to consider heavy peak loads at certain times, as for traffic to and from the New Jersey shore resorts over week-ends, to football games at Princeton, etc. The computation of the daily traffic, therefore, was based on the assumptions in Table 1; and it will be noted that, at times of peak traffic in one direction, it was assumed that the traffic on the other two lanes would be at half capacity.

Based on these assumptions, therefore, and on the desire to be quite conservative in all estimates, it was assumed that the maximum traffic would be 3 600 vehicles per hour, and the average, 54 000 per day, with a total of 18 360 000 per annum.

Economic Factors

Cost of Delays.—One factor of great importance economically is that of the cost of delays, due to interruptions at grade crossings with other highways, etc. This factor, of course, has not been considered in railway practice because of the obvious impracticability of stopping trains for these causes, although consideration has been given to the costs of delays of trains held at sidings for the passage of opposing trains on single-track lines, and the effect of this on the economics of operation.

Necessarily, the cost of delays must be measured, in the first instance, by a computation of the time lost by each vehicle and the total time lost by all vehicles delayed. It is obvious, also, that the delays at grade crossings will reach a maximum at times of peak loads, and be almost nil at hours of minimum traffic. At a drawbridge the amount of delays will be the time lost by the actual number of vehicles held up during openings.

TABLE 1.—ESTIMATED DAILY TRAFFIC, ROUTE 25.

Hour.	Percentage of maximum traffic.	CARS PER HOUR PER LINE.		Total cars per hour.
		1 and 2.	3 and 4.	
12:00-1:00 A. M.	70	840	420	2 520
1:00-2:00 A. M.	40	480	240	1 440
2:00-3:00 A. M.	10	120	60	360
3:00-4:00 A. M.	10	120	60	360
4:00-5:00 A. M.	10	120	60	360
5:00-6:00 A. M.	10	120	60	360
6:00-7:00 A. M.	10	120	60	360
7:00-8:00 A. M.	20	240	120	720
8:00-9:00 A. M.	30	360	180	1 080
9:00-10:00 A. M.	40	480	240	1 440
10:00-11:00 A. M.	50	600	300	1 800
11:00-12:00 A. M.	60	720	360	2 160
12:00-1:00 P. M.	70	840	420	2 520
1:00-2:00 P. M.	80	960	480	2 880
2:00-3:00 P. M.	90	1 080	540	3 240
3:00-4:00 P. M.	100	1 200	600	3 600
4:00-5:00 P. M.	100	1 200	600	3 600
5:00-6:00 P. M.	100	1 200	600	3 600
6:00-7:00 P. M.	100	1 200	600	3 600
7:00-8:00 P. M.	100	1 200	600	3 600
8:00-9:00 P. M.	100	1 200	600	3 600
9:00-10:00 P. M.	100	1 200	600	3 600
10:00-11:00 P. M.	100	1 200	600	3 600
11:00-12:00 P. M.	100	1 200	600	3 600
Total daily capacity (cars)				54 000

Due consideration also must be given to the loss involved by reason of the decreased use of the highway as a whole, or at least on the section affected by the delays. If, for instance, the normal capacity of 20 cars per lane per min. is reduced by reason of delays to 15 cars, the traffic capacity of the highway at times of peak demand is reduced by 25 per cent. Obviously, also, the cost of the delays must be calculated for the number of cars which can actually use the highway and not for the number which could use it if the traffic flow were free and uninterrupted.

In the case of a highway where the traffic demand is equal to the capacity of the highway any reduction in this capacity must be evaluated on the basis of the cost of the facilities so provided and not utilized, or on the basis of the cost of other facilities necessary to take care of the traffic not accommodated. On Route 25, it was estimated that if the traffic were delayed at a crossing at grade with another highway under such conditions that the traffic on the main highway could flow continuously for 3 min. and be interrupted for 1 min., the daily loss to the traffic on the two highways would be:

On the main highway	14 532 car-min. per day
On the cross street	9 000 car-min. per day
Total	23 532 car-min. per day

To be conservative and in view of all the conditions it was further assumed that the loss per annum would be at the rate of 7 000 000 car-min.

Assuming (on a basis to be developed) that a car-minute may be valued at 2 cents the yearly loss at such a crossing will be $140 000. This, capitalized at 6%, represents a value of $2 333 000. In addition, it was estimated that by reason of these delays, the traffic-carrying capacity was reduced by 12.1 per cent. If the cost of the highway be assumed to be $22 000 000, then this loss in efficiency can be calculated to be $2 662 000. The total amount, therefore, which might profitably be spent to eliminate a single isolated crossing under the conditions assumed would then be approximately $5 000 000.

Of course, if there were a group of crossings more or less close together, and if the traffic were properly regulated by a system of adequately controlled and synchronized signals, the economic loss due to the group would be only slightly greater than that due to one crossing only. It was assumed that the amount which could profitably be spent to eliminate a group of from two to five crossings would be as follows:

One crossing	$5 000 000
Two crossings	5 300 000
Three crossings	5 600 000
Four crossings	5 840 000
Five crossings	6 080 000

It will be obvious, of course, that this aspect, as in fact all other aspects, of this problem, must be studied carefully from its own individual point of view, taking into consideration the effect of all local conditions. The mathematical computations must necessarily be made and interpreted with a great deal of judgment.

Drawbridge Delays.—Route 25 had to cross the Hackensack and Passaic Rivers. The crossings of these rivers were studied by the writer for tunnels, and for bridges with movable spans having 35 ft. clear head-room above mean high water as required by the War Department. (Later, studies were made for fixed spans with 135 ft. clear head-room.)

The data on which calculations of the costs of delays at the drawbridges were based, were taken from the draw-tenders' records at adjacent bridges on the two rivers. These showed the number of openings, duration of each, and time of day. In using these data account was taken of the fact that the proposed bridges on the new line would have 35 ft. clear head-room above high water, when closed, instead of the 10 or 12 ft. of the existing bridges, and this requires fewer openings, as much of the river traffic could then pass without affecting the drawbridges. Future increases in the river traffic were estimated and allowance made therefor.

As in the case of grade crossings with other highways, two factors were taken into consideration: Actual losses due to delays of traffic, and losses due to decrease in capacity of the highway. In estimating the actual losses to vehicles, the average number of openings of the bridge for each hour of the day was applied to the number of cars estimated to use the highway during that hour, and the loss in car-minutes thus calculated.

These two items for the two bridges were evaluated as follows:

For traffic delays, 32 470 000 car-min. at 2.0 cents per min., $649 400 capitalized at 6%.........	$10 823 000
Loss of efficiency, 6.33% of $22 000 000.........	1 392 600
Total capitalized value of delays due to openings of the drawbridges.........	$12 215 600

Value of a Car-Minute.—Reference has been made to the value of a "car-minute", that is, the costs of delays per car per minute. Of course, this differs for each type of vehicle, but the general methods of estimating it are the same, except that for commercial vehicles there are more definite charges for delays which prevent the operation of this part of the owner's plant. It will be realized that a motor vehicle used for commercial purposes is just as much part of a manufacturing plant or selling organization as a machine in a factory, or an employee in a store. If idle when it might be working, it represents an item of loss.

For commercial motor vehicles, therefore, the stoppage is an element of expense. For non-commercial vehicles, however, the loss is not so clear, as, at first thought, it would seem to be a matter of comparative indifference whether a passenger arrived at his destination 2 or 3 min. earlier or later.

An experience with a taxicab in a traffic jam is, however, an illuminating example of what these delays actually cost; and if, for instance, one happened to be in a line waiting to cross the Hudson River by ferry, one would usually be willing to pay something extra for the privilege of moving up to the head of the line and avoiding the delay. The extra payments made to agents for theater tickets to avoid waiting at the box office, the rush for the first car of a train, etc., are all evidences of an innate realization of the costs of delays. The value of this economic factor of loss due to small delays has been doubted by many, not only laymen but engineers; nevertheless, it is a factor which must be given consideration.

These delays, of course, affect only some of the costs of operation. For purposes of analysis the average costs for a 3-ton truck for one year in the vicinity of New York were used for these estimates approximately as given in Table 2.

If a vehicle be delayed or stopped, certain expenses continue, such as Items Nos. 1, 2, 3, 4, 5, and 11. If during the delay the engine continues to run, there is also, of course, a certain consumption of gasoline and lubricants, but this will be small and has been omitted in the calculations.

Using the data in Table 2, and making similar allowances for light commercial vehicles and passenger cars used for commercial purposes, the following values for the cost of delaying a car for 1 min. were established:

For trucks	2.3 cents per car-min.
For light commercial vehicles	2.1 cents per car-min.
For non-commercial vehicles	1.0 cent per car-min.

It was further assumed that, for this particular highway, the economic values should be based on the assumption that the traffic would be divided in the following proportions:

	Percentage.
Trucks	50
Light commercial vehicles	25
Non-commercial vehicles	25

The average value of a car-minute (delays per car per minute), therefore, was assumed to be 2.00 cents.

TABLE 2.—Yearly Expense of Running 3-Ton Truck, New York City.

Item No.	Classification.	Amount.
1	Interest on $3 000 at 6%	$ 180
2	Depreciation, average life of 5 years	600
3	Insurance	300
4	Driver's wages, 300 days at $5.60	1 680
5	License fee	20
6	Gasoline, 6 miles per gal. at 25 cents	500
7	Lubricants	75
8	Tires	270
9	Repairs and maintenance	360
10	Miscellaneous	200
11	Overhead, 20%	837
Total		$5 022

Savings in Distance.—The economic effect of distance may be considered under two headings: Time, and operating cost. The value of time has already been studied in connection with delays at grade crossings and at drawbridges. It remains, therefore, to consider differences in operating costs due to increases or decreases of distance. These will be considered without reference to other elements, such as curvature, rise and fall, rates of gradient, or pavement surfaces. The cost of additional distance is affected, of course, quite appreciably by differences in types of pavement, but it can be assumed that the pavement will be of the same character on any two locations of the same route which may be under comparison, and that for highways of this type the pavement will generally be a smooth hard surface.

In computing operating costs not only are there several different classes of highways to be considered, but an almost bewildering variety of types and weights of cars, and almost as many degrees of ability in their operation and in conditions of efficiency. The determination of a cost unit, therefore, is not a matter of scientific accuracy. It has seemed possible, however, to develop average figures which are sufficiently accurate for the purpose in view, that is, the comparative operating costs of two locations on the same route on which the traffic is to be the same.

In the estimates for Route 25 a comparison of operating costs obtained from various sources indicated that, for the items affected, namely, depreciation; fuel; lubricants; tires; repairs; and miscellaneous (see Table 2), the following is probably a fair average cost of operation for the traffic expected to use this highway:

	Cents.
50% heavy trucks at 15 cents per mile........	7.5
25% light trucks and buses at 10 cents per mile	2.5
25% non-commercial at 6 cents per mile......	1.5
Average per mile..........................	11.5
Equivalent to	2.18 per 1 000 ft.

Assuming that 18 360 000 vehicles will use the highway annually, the decreased operating costs, due to a shortened distance of 1 000 ft., will be $400 248 per annum, which, capitalized at 6%, indicates that the sum of $6 670 000 might profitably be spent to save this distance.

Rise and Fall.—Results of experiments indicate that, because of certain deficiencies in the design of engines of motor vehicles, the cost of operation on moderate ascending gradients is no greater than on the level. While this may be true, it is perhaps reasonable to assume that such deficiencies may be corrected, and that the additional cost of operation on ascending gradients is that required to raise the load to a given height.

Experiments made at Iowa State College[3] indicated that the work done by a motor vehicle to raise a weight to a height of 1 ft. is approximately equal to the force required to propel the same weight over a level distance of 50 ft. on good pavements.

Taking the data previously used, it will be found that, for the average car on good pavements, the cost of producing power only, that is, fuel and oil, is approximately 3.22 cts. per mile, or about 0.03 cts. for 50 ft. Therefore, this may be assumed to be the cost of raising the average car (on this highway) 1 ft.

It is realized, of course, that, on highways as on railways, momentum will overcome certain smaller elevations without expenditure of power. It is considered further that the expenditure of power on light rates of gradient within certain limits of height may be negligible, but no information seems available to determine the limits within which this may apply. Arbitrary assumptions, therefore, were made in estimating operating costs due to rise and fall, but all these were believed to be conservative and to represent smaller, rather than greater, actual operating costs for this factor.

For a traffic of 18 360 000 cars per annum, the cost of 1 ft. of rise and fall at the rate of 0.03 cent per car is approximately $5 508, which, capitalized at 6%, represents a value of $91 800 as the amount which might profitably be expended to eliminate 1 ft. of rise and fall.

---

[3] "The Economics of Highway Grades," by T. R. Agg, M. Am. Soc. C. E., *Bulletin 65*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa, February 28, 1923.

However, after due consideration of all the factors involved—and it may be noted again that this is a purely arbitrary assumption—it was decided on this Route 25 that values for rise and fall should only be applied to gradients of 2%, or more (the maximum is 3.5%), and be varied for total lengths of continuous rise, as follows:

For total elevations of:	Profitable expenditure to reduce by 1 ft.
15 ft., or less	None.
18 to 20 ft	$30 600
20 to 25 ft	$61 200
25 ft., or more	$91 800

Curvature.—On Route 25, the minimum radius of curvature was fixed at 1 000 ft. It was considered that, for the proposed conditions, that is, a well-paved, non-skid (granite block) pavement, 50 ft. wide, curvature would have little or no effect on operating costs or capacity of the highway to carry traffic.

However, for highways with sharp curvature, where visibility is reduced, or conditions exist which slow up traffic, there will be costs due to delays and, in cases of maximum traffic demand, costs due to a decreased capacity of the highway to carry traffic.

Evidently, a certain amount of power is required to change the direction of a moving vehicle, but, so far as the writer knows, no experiments have been made to determine how much this may be for highway motor vehicles. Probably a reasonable estimate of the costs of delays, accident risk, and loss of capacity of any given highway would be a fair estimate of the cost of curvature or of the value of its removal. For the highway under consideration no account was taken of additional operating costs due to curvature.

DESIGN OF ROUTE 25

Application of Economic Theory to Actual Location.—In a general way two sets of factors determine the location of a new highway of the character herein described:

First.—The physical characteristics of the terrain and the need of reaching certain points for the benefit of the traffic and of the territory to be served; and

Second.—The economic factors affecting the operation of vehicles over the route.

The original legislative act authorizing the construction of this highway proposed it as an extension of old Route 1 which ran from Trenton to the westerly boundary of Elizabeth. This extension was, therefore, to run from Rahway Avenue, at the westerly boundary of Elizabeth, to the Jersey entrance to the Holland Tunnels in Jersey City.

On Fig. 1 is shown the original "Advisory Board" route and the location finally adopted for Route 25. It is worth while noting how physical and other conditions affected the location as well as how economic factors were applied to certain details.

Governing Points.—Doubtless if only through traffic traversing the whole length of the route were to be considered, in the quantity of 15 000 000 to 20 000 000 vehicles per annum, and if only the economic factors governing the cost of moving it were taken into account, economic justification could be shown for a practically straight, almost level, line from the Holland Tunnels to at least the easterly edge of Elizabeth, passing through Jersey City in a deep-level tunnel.

This latter, however, was considered entirely impractical, inasmuch as this highway was required to serve the requirements of Jersey City, making connections with its street system. Along the westerly side of the top of the hill (Fig. 1) there already runs a main north-and-south artery, the Hudson County Boulevard, with which connection also was desired. It was further necessary to provide for joining the new north-and-south State Highway (Route 1, 1927) which it was expected (in 1924) would come in from the north connecting the 178th Street Bridge across the Hudson with the proposed Bayonne-Staten Island Bridge.

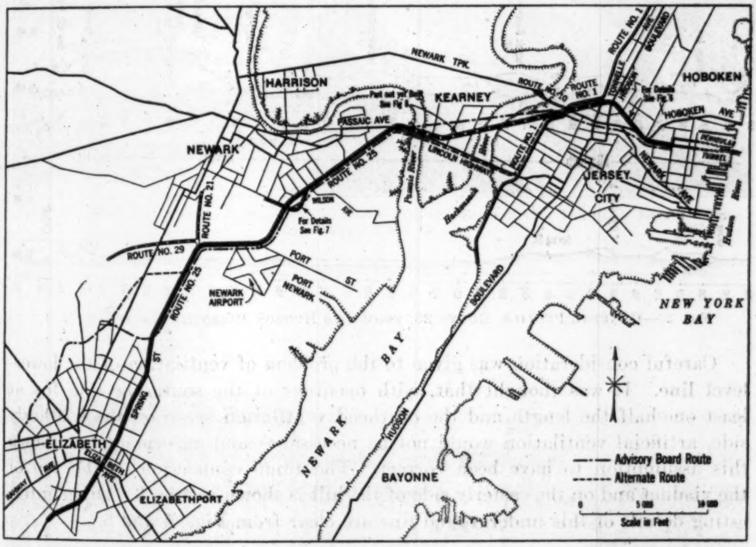

NEWARK TPK
HARRISON
KEARNEY
HOBOKEN
ROUTE NO. 1
AVE
BOULEVARD
ROUTE NO. 10
ROUTE NO. 1
TONNELLE
HUDSON
For Details See Fig. 9
HOBOKEN AVE
Part not yet Built See Fig. 8
PASSAIC AVE
PENNSYLVANIA TUNNEL
NEWARK
ROUTE NO. 25
LINCOLN HIGHWAY
Passaic River
ROUTE NO. 1
JERSEY CITY
NEWARK AVE
Hudson River
Hackensack
ROUTE NO. 21
WILSON AVE
For Details See Fig. 7
ROUTE NO. 29
PORT ST
PORT NEWARK
NEWARK AIRPORT
ROUTE NO. 25
BOULEVARD
NEW YORK BAY
BAY
ST
SPRING
NEWARK
HUDSON
ELIZABETH
ELIZABETH AVE
RAHWAY AVE
BAYONNE
ELIZABETHPORT
Advisory Board Route
Alternate Route
0
5000
10000
Scale in Feet

FIG. 1.—MAP OF JERSEY CITY, NEWARK, AND ELIZABETH, N. J., AND CONTIGUOUS TERRITORY, SHOWING LOCATION OF VARIOUS ROUTES.

These considerations determined the fact that the line would have to go over, instead of through, the hill, and an inspection of the profile, Fig. 2, will show that, in order to avoid crossings at grade with the Jersey City streets, crossings under the streets would probably be preferable to crossings over them. The maximum gradient of 3.5% on both sides of the hill just permitted the line to reach the elevation necessary to go under the streets. It also permitted a reasonable solution of the ramp connections to the main cross-streets (Fig. 3).

If, however, further justification were required from the point of view of economics of operation, it may be noted that, for an overhead line, the difference in elevation would have been approximately 40 ft., and this alone would have justified an additional expenditure of $91 800 × 40 = $3 672 000, for the lower-level line. No calculations were made in this instance to show the difference in construction cost between an overhead and an underground line, because it was obvious that the lower line permitted easier solutions of the problems of connection by ramps to the important north-and-south highways. Also, it was fairly evident that an overhead line could not have been built for $3 672 000 less than the underground line which actually was built. Furthermore, an overhead line would have been objectionable from an æsthetic point of view.

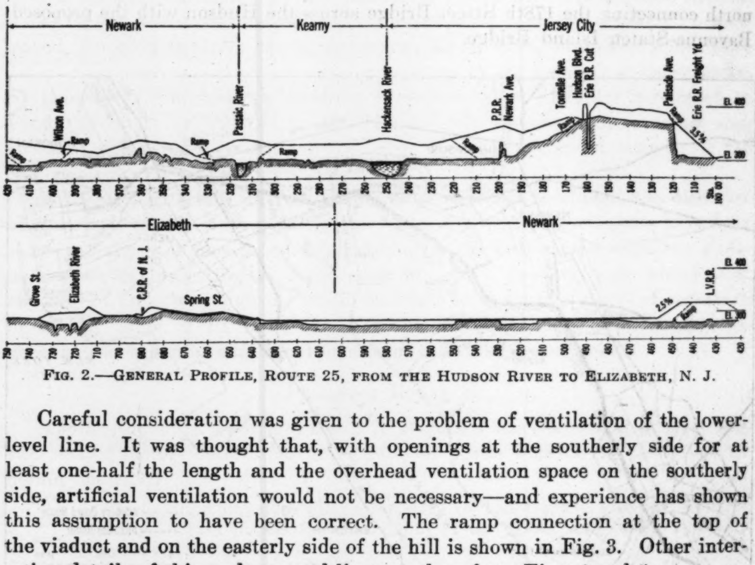

Newark
Kearny
Jersey City
Ramp
Wilson Ave.
Ramp
Ramp
Passaic River
Ramp
Hackensack River
Ramp
P.R.R.
Newark Ave.
Ramp
Tonnelle Ave.
Hudson Blvd.
Erie R.R. Cut
Palisade Ave.
Ramp
Erie R.R. Freight Yd.
3.5%
El. 400
El. 300
420 410 400 390 380 370 360 350 340 330 320 310 300 290 280 270 260 250 240 230 220 210 200 190 180 170 160 150 140 130 120 110 Sta. 100 00
Elizabeth
Newark
Grove St.
Elizabeth River
C.R.R. of N. J.
Spring St.
2.5%
Ramp
L.V.R.R.
El. 400
El. 300
750 740 730 720 710 700 690 680 670 660 650 640 630 620 610 600 590 580 570 560 550 540 530 520 510 500 490 480 470 460 450 440 430 420

FIG. 2.—GENERAL PROFILE, ROUTE 25, FROM THE HUDSON RIVER TO ELIZABETH, N. J.

Careful consideration was given to the problem of ventilation of the lower-level line. It was thought that, with openings at the southerly side for at least one-half the length and the overhead ventilation space on the southerly side, artificial ventilation would not be necessary—and experience has shown this assumption to have been correct. The ramp connection at the top of the viaduct and on the easterly side of the hill is shown in Fig. 3. Other interesting details of this underground line are clear from Figs. 4 and 5.

ALIGNMENT AT EASTERN END

Location Through Jersey City.—Of the various lines for the route through Jersey City (Fig. 6), the "Advisory Board" line was objected to by the authorities of Jersey City, because it was proposed as an open cut and it was felt that the two already existing open cuts, of the Erie and Pennsylvania Railroads, had created undesirable barriers across the city. Further study of the character of the line also indicated that the selection of Broadway (on the west side of the city) for the route was not entirely desirable.

The Erie Railroad Company objected to the use of its property on the south side of 12th Street because of certain plans it had for warehouse development there. This brought up questions of right of eminent domain as between

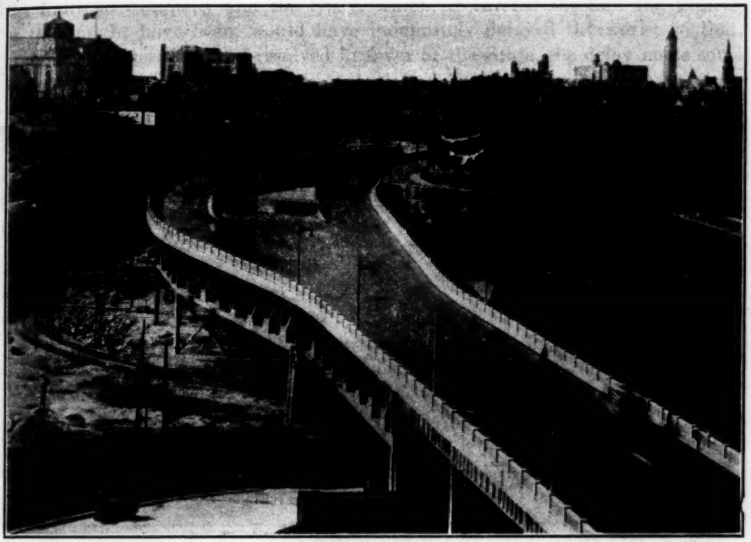


FIG. 3.—GENERAL VIEW OF VIADUCT, HUDSON COUNTY BOULEVARD, LOOKING NORTHWEST IN JERSEY CITY, N. J.

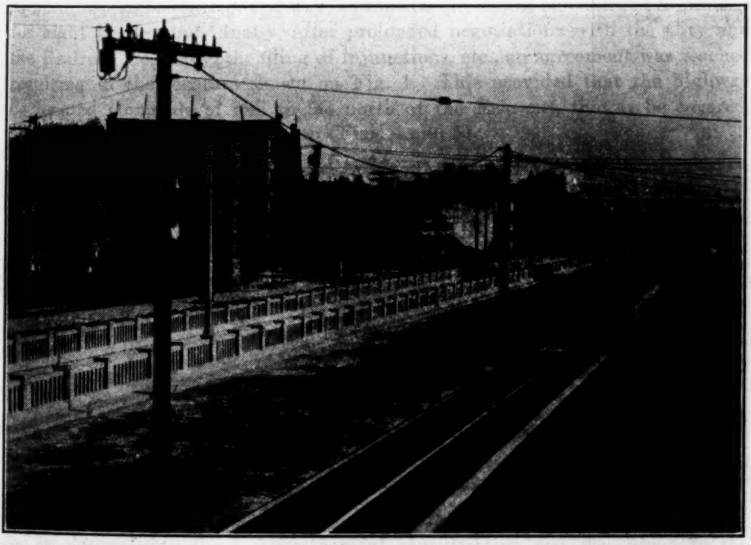


FIG. 4.—GENERAL VIEW OF COVERED CUT, HUDSON COUNTY BOULEVARD, JERSEY CITY, N. J., LOOKING EAST.

[illegible]

[illegible]

the Railroad Company and the State, which, if carried through the Courts, as they might have been, would have indefinitely delayed the work; so that, even had the question been resolved in favor of the State, the delay made some reasonable compromise desirable.

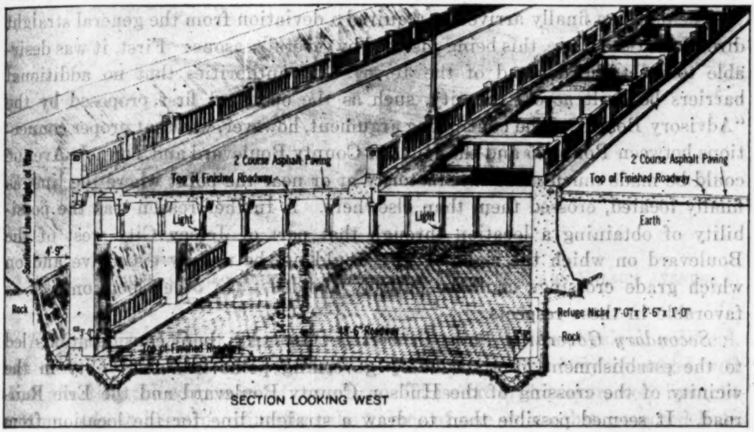

2 Course Asphalt Paving
Top of Finished Roadway
2 Course Asphalt Paving
Top of Finished Roadway
Light
Light
Earth
Southerly Line of Right of Way
4'-9"
Rock
Refuge Niche 7'-0" x 2'-6" x 1'-0"
Rock
Top of Finished Roadway
38'-6" Roadway
SECTION LOOKING WEST

FIG. 5.—TYPICAL SECTION OF DEPRESSED HIGHWAY THROUGH JERSEY CITY, N. J.

Studies were made, as shown by Fig. 6. The Railroad Company desired to maintain sufficient right of way, so that its cut might be enlarged to provide for eight tracks; but, finally, after prolonged negotiations with the City and the Railroad Company, the filing of injunctions, etc., an agreement was reached resulting in the location shown on Fig. 1. This provided that the highway be located alongside of and to the north of the Erie cut, that it be covered, and that it have a roadway on top (Figs. 4 and 5).

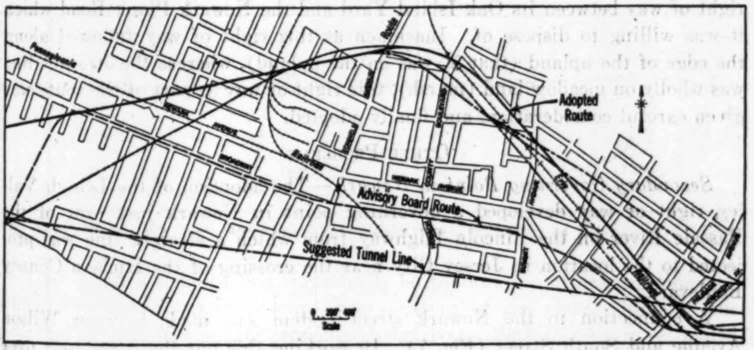

Pennsylvania
Route 1
Adopted Route
Railroad
Advisory Board Route
Suggested Tunnel Line
Erie
Railroad
Scale

FIG. 6.—STUDIES OF VARIOUS LOCATIONS THROUGH JERSEY CITY, N. J.

In determining the location careful consideration was also given to the elements involved in the destruction of rateable values by reason of buildings destroyed and land taken for highway purposes; also to the increment in valuations due to the creation of a broad boulevard crossing Jersey City (Fig. 4). All the governmental entities involved, City, County, and State, are vitally affected by these elements, and they exercise a considerable influence in the determination of locations.

The decision finally arrived at required a deviation from the general straight direction of the route, this being justified by several reasons: First, it was desirable to meet the demand of the Jersey City authorities that no additional barriers be built across the city, such as the open cut first proposed by the "Advisory Board". The controlling argument, however, was that proper connections between Route 25 and the Hudson County Boulevard and Tonnele Avenue could be made much more satisfactorily at or near the point where the line, as finally located, crossed them than elsewhere. A further reason was the possibility of obtaining a location through that part of Jersey City west of the Boulevard on which the right of way would not be unduly expensive and on which grade crossings could be entirely avoided. No other location was as favorable in these respects.

Secondary Governing Point in Jersey City.—All these considerations led to the establishment of a secondary governing point in Jersey City in the vicinity of the crossing of the Hudson County Boulevard and the Erie Railroad. It seemed possible then to draw a straight line for the location from this point to a point on the easterly boundary of Elizabeth.

Reconnaissances through Elizabeth had indicated this second point at approximately the location of the line as now built through Spring Street. This westerly objective, or governing point, however, was modified almost at once by the need of reaching the locality where Routes 21 and 29 now join this highway (Fig. 1). In some degree this was influenced also by the fact that the City of Newark had acquired and partly graded a right of way from near the airport to the Elizabeth line, which could be utilized for the highway.

It developed further that the Lehigh Valley Railroad Company owned a right of way between its Oak Island Yard and the Newark Plank Road which it was willing to dispose of. Inasmuch as this right of way followed along the edge of the upland (that is, was on the upland), whereas the straight line was wholly on meadow land (marsh), this right of way as part of the route was given careful consideration, and finally adopted.

OTHER PROBLEMS

Secondary Governing Point in Newark.—The adoption of the Lehigh Valley right of way developed a governing point in Newark just west of the Passaic River on the Lincoln Highway from which a straight line was projected to the location in Jersey City near the crossing of the Hudson County Boulevard.

A connection to the Newark street system was made between Wilson Avenue and South Street (Fig. 7). In working this out the State took over an existing contract between the City of Newark and the Lehigh Valley and Pennsylvania Railroad Companies which involved the abandonment of Port Street, thus permitting the development of the Oak Island Yards of these railways, which it crossed at grade, the State Highway at this point providing the principal means of access between the City of Newark, the Newark Airport, and Port Newark. This is an important example of effective co-operation between the local political and commercial interests, the railways, and the State, and between local and through traffic.

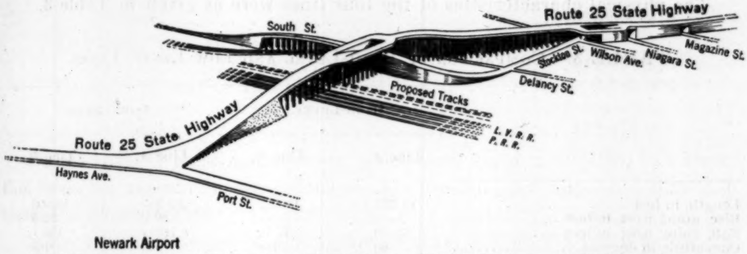

South St.
Route 25 State Highway
Magazine St.
Stockton St.
Wilson Ave.
Niagara St.
Delancy St.
Proposed Tracks
L. V. R. R.
P. R. R.
Route 25 State Highway
Haynes Ave.
Port St.
Newark Airport

FIG. 7.—STREETS, CONNECTIONS, AND RAMPS AT SOUTH STREET AND WILSON AVENUE, NEWARK, N. J.

Factors Other Than Economic.—In considering all the physical, political, and other factors described, it may be felt that they entirely overshadowed those affecting the economics of operation. These latter, however, were brought continuously to bear in estimating the values of one or another proposed line. The writer is firmly convinced that these economic aspects are important elements which no conscientious engineer can afford to ignore, but the other elements have been set forth to emphasize that both aspects must be given due regard and their relative values properly appreciated.

Application of Economic Factors.—As an indication of the use to which these economic factors were put, there are cited here some comparisons made of two lines which were projected between the Hudson County Boulevard and the west side of the Hackensack River.[4] These Lines *A* and *B* are shown on Fig. 8 and the various profiles studied on Fig. 9.

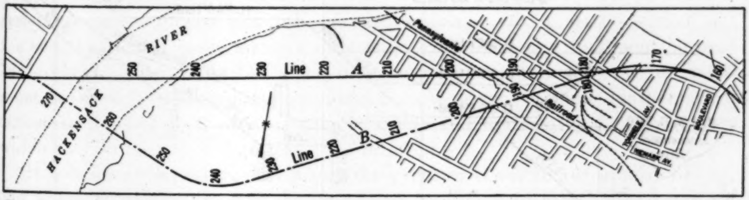

RIVER
HACKENSACK
Line A
Line B
270
260
250
240
230
220
210
200
190
180
170
160
Pennsylvania Railroad
Tonnelle Av.
Newark Av.
Boulevard

FIG. 8.—PART OF ROUTE 25, SHOWING ALTERNATIVE LOCATIONS ACROSS HACKENSACK RIVER.

Line *A* was a straight line (practically on the location afterward adopted), which, however, required a long skew crossing of the river. Line *B* was deflected to provide a square crossing of the river. Both lines were estimated for high and low levels.

---

[4] These estimates were made in 1924 and 1925 and applied to conditions assumed at that time. Later, in 1929, conditions, especially in regard to the War Department requirements for a bridge crossing on the skew line, made it necessary to revise the estimated costs of construction.

The high-level lines provided for a viaduct structure from the exit of the crossing under the Hudson County Boulevard, over the Pennsylvania Railroad to a bridge over the river with 35 ft. clear head-room above mean high water.

The low-level lines were in an open cut from the Boulevard under-crossing, under the Pennsylvania Railroad, and with tunnels under the rivers.

The physical characteristics of the four lines were as given in Table 3.

TABLE 3.—COMPARISON OF HIGH-LEVEL AND LOW-LEVEL LINES.

	HIGH-LEVEL.		LOW-LEVEL.	
	Line *A*.	Line *B*.	Line *A*.	Line *B*.
Length, in feet	12 253	13 003	12 253	13 003
Rise, going west, in feet	......	......	86.2	83.9
Fall, going west, in feet	35.70	35.70	160.4	158.1
Curvature, in degrees	60	168	60	168

The estimated costs of construction were:

High-Level:	
Line *A*	$8 898 410
Line *B*	8 085 780
Low-Level:	
Line *A*	$11 694 223
Line *B*	11 099 869

The difference in the lengths of the two lines is about 750 ft., and it will be recalled that the justifiable expenditure to save 1 000 ft. in distance on this line, and for its expected traffic, was estimated at $6 670 000. It is clearly evident, therefore, that, for either level, Line *A* is preferable to Line *B*.

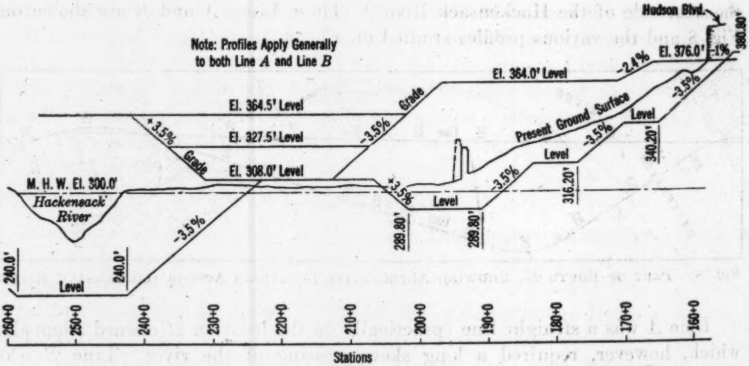

Note: Profiles Apply Generally to both Line A and Line B
Hudson Blvd.
380.80'
El. 376.0'
-1%
-2.4%
El. 364.0' Level
-3.5%
Grade
El. 364.5' Level
+3.5%
-3.5%
El. 327.5' Level
Grade
El. 308.0' Level
M. H. W. El. 300.0'
Hackensack River
-3.5%
+3.5%
Level
289.80'
289.80'
Present Ground Surface
-3.5%
Level
-3.5%
Level
340.20'
316.20'
240.0'
240.0'
Level
260+0
250+0
240+0
230+0
220+0
210+0
200+0
190+0
180+0
170+0
160+0
Stations

FIG. 9.—PROFILES SHOWING STUDIES FOR LINES CROSSING HACKENSACK RIVER.

There was then to be made the comparison between the high and low levels (bridge or tunnel) for Line *A*. For the low level there was added to the construction cost the capitalized value of the added rise and fall, and to the high-level line the added operating costs due to delays to be caused by the opening of the drawbridge. The comparison of the two lines on this basis, therefore, was at that time worked out as follows:

High level and bridge: Estimated costs of construction	$8 898 410
Capitalized cost of delays at bridge	8 000 000
Total	$16 898 410
Low level and tunnel: Estimated costs of construction	$11 694 223
Capitalized cost of rise and fall	9 780 000
Total	$21 474 223

On the basis of these assumptions, which were made in 1924-25, in connection with the earliest investigations of this problem, the high-level line then seemed to be preferable.

Since writing this paper, the profile of that part of the route across the Valley of the Hackensack and Passaic Rivers between Jersey City and Newark has been changed so as to provide 135 ft. clear head-room above mean high water at the river crossings. This change was brought about as the result of conferences to settle the long controversy as to the relative merits of tunnels for these crossings as compared with drawbridges with 35 ft. clear head-room. The writer is entirely in accord with this final decision.

Modern Highway Demands

It has been impossible within the limits of a paper of this kind to fully demonstrate all the mathematical processes by which the data adopted for the studies for Route 25 were developed; but it is thought that the brief outline of their application to one section of the route will serve as a sufficient indication to experienced engineers, and no others should attempt to apply or develop them.

Highway traffic is so varied in the number and kind of vehicles which compose it, and operating costs are so different in different localities, that the resistance values of rise and fall, curvature, delays, etc., must be worked out for each particular case; and not only must experience and judgment be used in developing these data, but equal or even greater experience and judgment must be used in determining the weight to be given to these mathematical demonstrations of costs of resistance to traffic and the local and other factors which may influence or determine the location.

It is believed, however, that where the problem is wholly, or almost wholly, one of transportation, as was the governing condition in locating and designing Route 25, these resistance values should be worked out, so that they may be available as at least one factor to be considered in determining the location or in deciding between two or more possible variants.

With highways costing $3 000 000 to $5 000 000 per mile, as was the case with the first 8 miles of Route 25, engineers owe it to their clients or employers to make these demonstrations, and the governing bodies charged with carrying out works of this nature owe it to the taxpayers, not only to see that, for instance, the concrete is properly proportioned, the steel properly designed and erected, and the pavements adequate and properly laid; but before all this, to ensure that the general design of the route—that is, its location, both in plan and profile—is such that there is a true economic proportion between the costs of its construction and the costs of operation of the vehicles which have to use it.

ACKNOWLEDGMENTS

The writer is indebted to the late A. M. Wellington, M. Am. Soc. C. E., and his "Economic Theory of Railway Location" for the basis of the theory herein applied to the location of Route 25.

The work was carried out under the general direction of the New Jersey State Highway Commission; Gen. Hugh L. Scott, U. S. A. (*Retired*), Chairman; and William G. Sloan, M. Am. Soc. C. E., State Highway Engineer. Great credit is to be given to all of these for their far-sightedness in the original conception of the general features of this highway and their initiative, co-operation, and help in the solution of the problems involved, both these herein referred to and innumerable others.

The writer also gratefully acknowledges the assistance of S. Johannesson, M. Am. Soc. C. E., who not only worked out the mathematical solutions of these problems, but who also was in entire charge, under the direction of the writer, of the design of the structure.

DISCUSSION

W. W. Crosby,[5] M. Am. Soc. C. E. (by letter).—The writer has endeavored in the past to point out the fundamental significance of the proper location of highways,[6] Mr. Lavis adduces an outstanding example of the sums involved, and the general interest therein may be proportional to their magnitude.

The author states that,

"* * * for handling large volumes of traffic through congested areas, highways should be designed [and paid for] for the main purpose of transportation rather than as adjuncts of the abutting properties." * * * where congestion of existing city streets interferes with the reasonably free passage of vehicles and pedestrians, highways of the type described are necessary not only to expedite through traffic, but also to restore such streets to their normal uses."

When these statements become generally appreciated, the highway will begin to be established in its proper place as an "element of transportation" comparable to the railway or canal.

It is true that a beginning has been made toward this end. The State highways of Pennsylvania have been largely located (or relocated) since 1924 with full recognition of the principles suggested by Mr. Lavis, although perhaps no single example of remarkable magnitude can be cited. Other States have attempted location more or less along similar lines.

However, the economist is not, and never will be, entirely free from the politician in this field. Perhaps this is just as well because the politician represents the sentimental side, and highways are not after all intended entirely for material purposes.

There are "intangibles" connected with any highway improvement that should not be overlooked even in its location. For example, one can estimate very closely the need for the width of a roadway to accommodate traffic, but to that width must be added, more or less arbitrarily, some further width for uses other than vehicular. Furthermore, it has been proved unwise to be too "economical" in that addition. Mr. Lavis refers to "Factors Other than Economic". "A demand for a new type of road" and a "Need of Special Highways" are also suggested by the author.

At the International Road Congress at Milan, in 1926, the Italian delegates proposed the endorsement of the idea of building such special highways where traffic conditions might justify them (as in certain localities in Italy). They even suggested making them toll roads, provided the eventual return of such roads to the State was assured. The American delegates, probably fearful from their experiences with toll roads in the past, united with the British to block the passage of this resolution. However, this country is actually building similar roads now whether or not the toll factor is evident.

To criticize fairly the location of "Route 25", described by Mr. Lavis, is impossible for the writer at present. Apparently, its alignment is question-ably indirect, although Mr. Lavis seems to have anticipated this comment by his explanations. Probably here the "political" influence has swayed the economic decision. There seem to have been certain "legislative controls" and the influence of other route connections is mentioned as affecting the line. It is not quite clear, however, why the main route was swung aside to meet these other roads instead of bringing them to it. Neither is it clear to the writer why a roadway width of 50 ft. was selected. Do not most highway authorities condemn an odd number of lanes?

---

[5] Cons. Engr., Coronado, Calif.

[6] *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 1007.

That "time is money" is now generally recognized by transportation authorities, and the author's exposition of the "Value of a Car-Minute" is commendable. His assumptions as to the division of the traffic on the highway are interesting, especially because of their apparent departure from the figures for New Jersey traffic reported in times past by the State Highway Commission. The writer believes that the assumed proportions of heavy trucks, 50%, light trucks, 25%, and non-commercial vehicles, 25%, are almost the reverse of the earlier reported figures.

The author proceeds to use operating-cost figures that may be interestingly compared with those used in Pennsylvania.[7] He concludes his all too brief paper with a statement of "Modern Highway Demands". The writer scarcely needs to reiterate his position in this matter, but he does wish to felicitate Mr. Lavis on the forcible expression of his conclusions, which seem incontrovertible.

WALTER LORING WEBB,[8] M. AM. SOC. C. E. (by letter).—This paper demonstrates the possibility, and even the practical necessity, of applying the principles of economics to the construction of through highways, that new type brought into existence by the motor car—a highway designed to serve the traffic which desires to travel fifty miles or more without stopping and which is best served by avoiding all intermediate cities or obstructions of any kind. When the late Arthur M. Wellington, M. Am. Soc. C. E., brought out his book entitled "Economic Theory of the Location of Railways", he dealt with a subject which seemed very complicated to those who tried to master its principles, and yet Wellington had a subject which was comparatively simple. A railway has the right of eminent domain and can locate its line as it pleases; it has a private right of way on which no one else can travel; it does not stop its trains for highway crossings, and it is only a matter of finance to avoid grade crossings, and possible stops and delays, at intersections with other railways.

It is a comparatively simple matter to reconcile existing physical conditions with the conditions necessary for the economical movement of trains when a railway company has the broad powers of eminent domain and the absolute control of its own right of way; but the problem, as exemplified by the construction of Route 25, described by the author, is infinitely more complicated. The State, several municipalities, and several railroad companies—each with their several powers of eminent domain and their several conflicting interests, each jealous of their powers to control matters to serve their own special interests—must be harmonized as nearly as possible. When one tries to introduce the added complication of designing a motor highway which will make possible the economical operation of motor vehicles, the problem becomes so complicated that it requires the highest grade of engineering skill and experience for its successful solution.

---

[7] "Highway Location," p. 134 *et seq.*

[8] Cons. Engr., Philadelphia, Pa.

However, it should be realized that this last mentioned element—the economical operation of motor vehicles—is a vital and essential element in the solution of every such problem. In an address[9] before the Engineers' Club of Philadelphia, Dr. George Otis Smith, Director of the U. S. Geological Survey, emphasized the necessity for the modern engineer to be an economist, or, in other words, that he must avoid all features of construction which necessitate methods of operation that are avoidably wasteful. In so far as the design for such a motor-highway project fails to provide for such economies of operation as are possible, to just that extent the design is a failure and is a waste of the several millions of dollars which are spent on it.

This subject is becoming increasingly important, since the construction of Route 25 in New Jersey is one of the first of many similar projects which will undoubtedly arise in the solution of the fast growing motor-transport industry.

The writer is aware that he has said little or nothing that has not already been well said by Mr. Lavis, but he believes that the matter is so important that it should be reiterated and emphasized until the general public, as represented by State and municipal authorities, who do not look at these matters from the engineer's standpoint, become more conscious of its importance as a matter of public welfare and economy.

It is unfortunate that the data on the actual cost of motor operation, the cost of applying brakes and resuming speed, the cost of stops and starts, and the effect of grade and curvature, are so meager. Undoubtedly, there will be an accumulation of such data in the near future. The necessity for the economical operation of motor-bus and motor-truck lines will probably result in the scientific study of those elements by the managers of those lines, and engineers will probably have a much larger accumulation of data on which to base their calculations than is now available.

Although the writer has no experimental data on the subject, he is of the opinion that curvature, even "easy" curvature, is not as harmless as it is commonly supposed to be. The centrifugal force which must be resisted in some way when a vehicle passes around a curve is measured by the formula, $C = \frac{W v^2}{g r}$. Its magnitude may be estimated by assuming numerical values as follows:

Let W = the weight of a car, or truck, assumed numerically as 15 000 lb.;
v = the velocity, assumed as 30 miles per hour, or 44 ft. per sec.;
g = the acceleration of gravity, or 32.2 ft. per sec. per sec.; and,
r = the radius of curvature, here assumed as 1 000 ft.

Then, C, the centrifugal force = 902 lb.

---

[9] *Engineers and Engineering*, July, 1930.

This stress must be taken up by the friction of the tires on the pavement, except as it would be possible to counteract it, at least partly, by elevating or banking the curve on the outside. In this numerical case, reducing the radius from 1 000 to 300 ft. and still maintaining the velocity at 30 miles per hour, would increase the stress from 902 to more than 3 000 lb.

It would probably be very difficult to determine experimentally the increase in rate of tire wear due to curvature. Part of the deterioration will be due to actual skidding or scraping of the tire on the pavement, thus wearing down the tire shoe; but some will be due to mere stress which, when long continued and repeated, will wear down the resistance even of the rubber, and cause its earlier destruction.

The objection to curvature may be very slight or even negligible, from the standpoint of loss of time or required increase in motive power—and hence in the cost of gasoline and oil—but the effect on tires and on the internal stresses in the car frame, which will cause more rapid deterioration, is certainly very appreciable, even if it is difficult to evaluate.

It is to be hoped that, when problems similar to those involved in the design of Route 25, are brought before the State or municipal authorities concerned, those elements of economy of operation will be thoroughly studied and will be given their due weight in making the final decision.

Of course, every such problem involves conflicting interests of greater or less importance. The public must insist that less important interests must give way (and perhaps be compensated) for a design which will best serve the interests of the majority. The danger lies in the possibility that powerful financial or political groups, whose real financial or public interest is less important, will be able to control the situation.

The profession owes much to Mr. Lavis for his practical demonstration of the possibility of the scientific application of Wellington's fundamental principles to such a complicated problem, and also for his demonstration that it is only by the use of such a general method that such problems can be properly solved.

JAY DOWNER,[10] M. AM. SOC. C. E. (by letter).—This description of the Holland Tunnel-Elizabeth Highway, known as Route 25, is a valuable exposition of the new conception of highway design and construction. In recent years marked improvements have been made in such component elements as grades, alignment, bridge structures, and paving materials. The outlook of planners and designers has now expanded beyond these primary essentials to include two other important elements of comprehensive design. These are the isolation of rights of way from abutting private property and the elimination of grade crossings.

The application of these last named principles in congested regions, such as that traversed by Route 25, even at very heavy costs, becomes justified when overbalanced by the economic pressure of losses due to traffic delays. The rapidly increasing volume of mechanical transport on roads in the United

---

[10] Chf. Engr., Westchester County Park Comm., Bronxville, N. Y.

States has made evident, the analogous relation of the trunk-line, express highway to the railroad.

Through congested city sections, costly viaducts or tunnels are the principal means of accomplishing both isolation from abutting property and grade separations. In areas where vacant land is still available, this isolation as well as grade separation structures and ramps can be attained more economically by the acquisition of rights of way wide enough to afford flanking strips on both sides of the paved roadway.

This principle has been followed in laying out the Westchester County Parkways. The minimum width adopted for these parkways is 200 ft. and 300 ft. is desirable. The paved roadways which have been laid up to the present time (1930) are 40 ft. wide, but the Hutchinson River Parkway and the Saw Mill River Parkway have been sub-graded to a width of 60 ft. to provide for an additional 10-ft. lane on each side of the initial 40-ft. pavement.

The character of the greater part of the area traversed by Route 25 doubtless precluded the application of the excess right-of-way principle, but in laying out trunk-line express highways there are doubtless broad fields offered for its application, even in the environs of many of the large cities.

Wherever it can be applied, this principle will doubtless commend itself to the designers of such highways, both for the economies in grade-crossing eliminations and the opportunities afforded for the amenities of parkway treatment applied to the flanking strips separating the paved roadway from abutting private property or subsidiary streets. The cost of excess land area may be easily outweighed by lower grade-separation costs and the very tangible economic as well as esthetic value of parkway treatment.

Through metropolitan areas of residential character, the effect of a trunk highway on near-by land values may be of greater economic importance to the immediate region than its value as a transportation route. The parkway type of express highway route invariably enhances near-by property values in contrast with the depression of values that has been experienced in many places along heavily traveled, modern roads in direct contact with private property.

S. JOHANNESSON,[11] M. AM. SOC. C. E. (by letter).—The author states that the highway considered in his paper will cost from $3 000 000 to $5 000 000 per mile. When the length of such highway is measured in miles rather than in feet, the total expenditure becomes so great that the first reaction is likely to be expressed in the question: Is it worth its cost? An affirmative answer is given in the paper, but it is of sufficient importance to warrant a re-statement.

The ultimate purpose of the highway is to decrease the cost of transportation of people and merchandise between the points served by it. This might also be expressed in other terms; for example, it might be said that the reason for building the highway is to relieve the congestion of existing traffic facilities or to decrease the time required to travel from one point to another. These statements are, in fact, only different ways of expressing the same definition.

It is probable that not less than 30 min. is saved in traveling over the new highway from the Holland Tunnel Plaza in Jersey City to the airport at

---

[11] Designing Engr., State Highway Comm., Jersey City, N. J.

Haynes Avenue, Newark. Using the data given in the paper, the traffic is estimated at 18 360 000 vehicles per annum and is valued at 2 cents per car-min. Therefore, the capitalized value of the savings in cost of transportation would be $183 600 000, or, say, $23 000 000 per mile of highway. It is apparent, therefore, that an expenditure of $3 000 000 to $5 000 000 is entirely justifiable, because it permits a saving several times the cost of construction.

The determination of the feasibility of a new highway should properly be based on an economic balance between the cost of producing the new facilities and the saving gained by them in cost of transportation. Generally speaking, this is directly proportional to the volume of traffic. Consequently, the amount of money that may be spent economically on new highway projects is directly proportional to the volume of traffic. When the traffic is heavy, as in the case considered by Mr. Lavis, this amount may often be found to be surprisingly large.

D. P. Krynine,[12] M. Am. Soc. C. E. (by letter).—Modern technology and economics are intimately bound together; this statement is perhaps more true of Highway Engineering than of any other technical branch since the cost of a product moving from the producer toward the consumer is often considerably increased by bad road conditions. Consequently, the necessity of an "economic design" arises, that is, a design in which the needs of transportation are met in the most efficient and, at the same time, the cheapest way.

Nowadays, American engineers build roads in practically all parts of the world. Therefore, it is important for them to take into consideration the economic features of a given country or region in order to be able to design such roads effectively.

It may be observed that, in its economic development, a country or a region passes through the stages of being: (*a*) A colony or undeveloped country, such as America was about 1730; (*b*) a developed country, such as America in 1890 or 1900; and, (*c*) a congested area, such as New Jersey or New York at the present time (1930).

Each of these three steps may be found in different countries of this epoch. In the beginning of Period (*a*) there are practically no roads in a country or a region. Gradually, lanes or earth roads appear that satisfy the elementary needs of the settlers. The road system develops little by little, being adapted to the demands of growing industries. However, such a development of a road system involves technical errors, especially in properly selecting the right of way; and often subsequent generations are obliged to waste considerable sums of money in rectifying such errors.

Obviously, such a situation as that which existed about two centuries or more ago, when road experts had but poor ideas concerning highway technique and economics, cannot be tolerated to-day. A highway designer who deals with the road system in an undeveloped country, should prepare a comprehensive plan of construction. A thorough economic study of the region must precede any technical road survey; the existing volume of traffic should be determined, and the future volume should be estimated. Then, a proper type of each link of the system should be selected, almost in the same manner as is done by a structural engineer when determining cross-sections of a bridge according to the expected stresses. In highway design the stresses correspond to the volume of the traffic, and the cross-sections to the type of the road.

---

[12] Research Associate in Soil Mechanics, Dept. of Civ. Eng., Yale Univ., New Haven, Conn.

It must be borne in mind that in an undeveloped country or region the road system is a powerful means of controlling the development of industries, and a foreign consulting engineer called to work in the design of such a road system should have a series of conferences with Government officials and economists.

In an undeveloped country or colony the highways are indubitably elements of transportation. In time, however, railroads and other perfected means of transportation appear and, to them, highways cede their rôle of a means of through transportation. Thus opens an epoch during which many (perhaps a majority) of roads and highways become merely municipal features like the water supply or the fire department.

The third stage is that so brilliantly described by Mr. Lavis. The construction of Route 25 in New Jersey is an introduction into the transportation system of a new kind of link that is something between "highway" and "railway." This new member of the transportation family may be called "super-highway" or "autostrade", as in Italy. Incidentally, Italy has constructed the following autostrades to date: (1) From Milan to Lakes Maggiore and Como; length, about 60 miles; width, 33 and 52 ft.; (2) from Rome to Ostia (a short distance); and, (3) from Naples to Pompeii (a short distance), and a vast program is under construction and consideration.[13]

HAROLD M. LEWIS,[14] M. AM. SOC. C. E. (by letter).—The author has described a modern highway as one which is designed with low grades and curvature and with a maximum elimination of delays from crossings of intersecting waterways, main highways, and local streets. Such is the "express highway" which has come to have an important place in the design for highway transportation in any large metropolitan area. New Jersey State Highway Route No. 25 was a pioneer project in such construction and has fully justified the faith of those who designed it.

Types of Express Highways.—That "express highways" are needed seems to be admissible without further discussion, but just what form such routes should take and what their relation should be to the other parts of the highway system and to the development of the adjoining land are questions requiring still further study. That there are various solutions of such problems is shown by examples already available. The "super-highways" radiating out from Detroit, Mich., on 204-ft. rights of way illustrate a type which combines the express highway with a main artery for rail transportation, facilities for local traffic, and the intensive development for business purposes of most of the fronting property. The West Side Elevated Highway, along the Hudson River waterfront of Manhattan, in New York City, utilizes a portion of the air rights above the wide open space composed partly of public streets and partly of the marginal wharf or place under the jurisdiction of the Dock Department. The space beneath this highway is still available for street traffic, except where ramps interfere. In no other place in Manhattan could such an elevated highway have been constructed without acquiring large areas, or paying damages to abutting buildings for taking away their light and air.

---

[13] See "Tabula viarum Romani Imperii," Rome, 1930.

[14] Cons. Engr.; Engr., Regional Plan of New York and Its Environs, New York, N. Y.

The City of Chicago has proposed an Avondale Super-highway, 10 miles long, from the heart of the city to its northwestern limits. This highway would parallel the tracks of the Chicago and North Western Railroad and would be elevated above the normal street system to which connections would be provided at intervals of about 1 mile, planned so that left-hand turns would be unnecessary. There would be an adjacent local low-level street, 50 ft. wide, for 9 of the 10 miles of the improvements.

The City of New York is planning a cross-town express highway in Manhattan to be constructed as an extension of a mid-town vehicular tunnel under the East River and to connect with a second vehicular tunnel under the Hudson River projected by the Port of New York Authority. This express highway will be a deep-level tunnel passing beneath all existing or projected rapid transit subways and will have no connections with the street system, except at the two ends—one near Second Avenue on the East Side and the other near Tenth Avenue on the West Side.

In Westchester County, New York, the County Park Commission has planned an express highway for general traffic to be constructed on a new right of way roughly parallel to the New York, New Haven, and Hartford Railroad. The right of way has been acquired, but to date (1930), construction has not been started.

In the Detroit District a 40-ft. elevated express toll highway has been proposed above the Detroit-Pontiac Line of the Grand Trunk Western Railway of the Canadian National System, as part of a \$100 000 000 electrification and improvement program. The roadway would be 40 ft. above the rails, and intersecting highways would cross on an intermediate level between the railroad and highway. It is proposed to build the highway in sections and to establish the toll on zone rates.

The foregoing examples indicate the various forms which express highways are taking. In addition, there is the "parkway" which may provide an "express" highway restricted to passenger car traffic. Not all parkways may actually be express highways as some are primarily for local recreational purposes, but when—as in the case of the Bronx River, Saw Mill River, and Hutchinson River Parkways, in Westchester County, New York—they form main highways to or from metropolitan centers, they are closely related to such general traffic highways as those discussed herein.

Proposed Express Highway System for New York Region.—The Graphic Regional Plan prepared for New York and Its Environs included a comprehensive system of express highways, of which New Jersey State Highway Route No. 25, and the West Side Elevated Highway and the cross-town tunnel in Manhattan, form important parts. For smaller regions, or those with less geographical complications, it might not be necessary for the express highway system to cross the main business center, but only to skirt its edge. In this case, however, two lines were shown across the main center in Southern Manhattan. One of these utilized the Holland Tunnel and the other the projected mid-town crossings previously mentioned.

North and south routes were proposed on both the main water-fronts of Manhattan and also on both sides of the Hackensack Meadows, in New Jersey, and a fifth route through Brooklyn and Queens Boroughs parallel to the East River. A total of six radial routes was shown, only one of which (the route described by Mr. Lavis) was extended to the edge of the region. Route No. 25 was officially projected to Philadelphia, Pa. The other five routes extended out from 20 to 25 miles from the metropolitan center. Fig. 10 shows two additional express highways proposed by the New Jersey State Highway Department. One of these is a branch of Route No. 25 through the center of Newark, the other would lead to the west across the congested areas of Essex County.

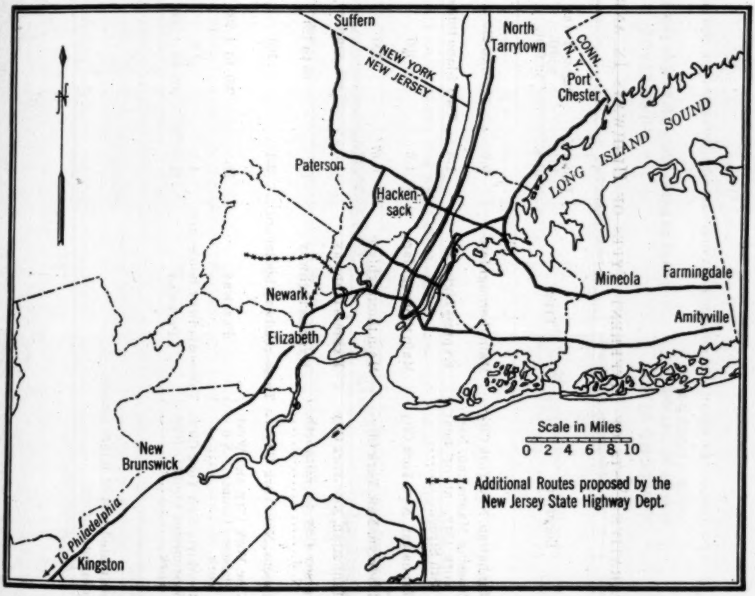

Suffern
North Tarrytown
NEW YORK
NEW JERSEY
CONN.
N.Y.
Port Chester
LONG ISLAND SOUND
Paterson
Hackensack
Newark
Elizabeth
Mineola
Farmingdale
Amityville
Scale in Miles
0 2 4 6 8 10
Additional Routes proposed by the New Jersey State Highway Dept.
New Brunswick
To Philadelphia
Kingston

FIG. 10.—PROPOSED SYSTEM OF EXPRESS HIGHWAYS FOR NEW YORK AND ITS ENVIRONS.

Comparative Costs.—The cost of such a highway through intensively developed areas of high land values is tremendous, particularly if it becomes necessary to locate it entirely above or below the level of the existing street systems. In suburban territory, an express route may follow the surface of the ground throughout most of its length, and grade separations may be installed only at the main intersecting highways; secondary streets can terminate in part at the highway and in part cross by short offsets along the line of the express highway.

TABLE 4.—COMPARATIVE COSTS OF DIFFERENT TYPES OF HIGHWAYS IN AND ADJACENT TO NEW YORK CITY.

Name of highway.	Location.	Type.	Length, in miles.	Right-of-way width, in feet.	Cost per mile.	Remarks.
Sixth Avenue Extension.	Manhattan, New York City	Main thoroughfare	0.6	100 to 210	$14 000 000†	Opened 1930; forms new approach to Holland Vehicular Tunnel.
Mid-town Tunnel..........	Crossing Manhattan, New York City, at or near 38th Street..............	Express highway	1.5	Two 2-lane tunnels	11 200 000*	Final plans being prepared.
Varick Street Widening and Seventh Avenue Extension..............	Manhattan, New York City	Main thoroughfare	1.3	100	6 300 000	Opened 1918; Varick Street widened from 65 to 100 ft.
Flatbush Avenue Extension....................	Brooklyn, New York City.	Main thoroughfare	0.67	100	4 450 000	Approach to Manhattan Bridge; opened 1909.
West Side Elevated Highway......................	Manhattan, New York City	Express highway	1.5	70-ft. viaduct	4 300 000	Section, Canal to 23d Street only; opened 1930; no right-of-way costs required.
New Jersey State Highway No. 25..............	Jersey City to Elizabeth, N. J....................	Express highway	13.	66 to 120	2 200 000*	Partly completed; remainder under construction.
Nassau Boulevard........	Queens, New York City...	Three-roadway boulevard	2.1	160	1 171 000‡	Section, Strongs Causeway to Fresh Meadow Road; includes paving of only a 30-ft. roadway.
Bronx River Parkway....	New York City and Westchester County, New York....................	Parkway	16.	200 to 1 200	1 060 000	Final section opened in 1925.
Kings Highway...........	Brooklyn, New York City.	Three-roadway boulevard	4.1	160	740 000	Opened 1926; includes paving of central roadway only.
Hutchinson River Parkway......................	Westchester County, New York.....................	Parkway	11.	200 to 1 000	370 000	Opened 1928.

* Preliminary estimates.
† Acquisition cost plus estimated cost of improvement.
‡ Estimate based on incomplete figures.

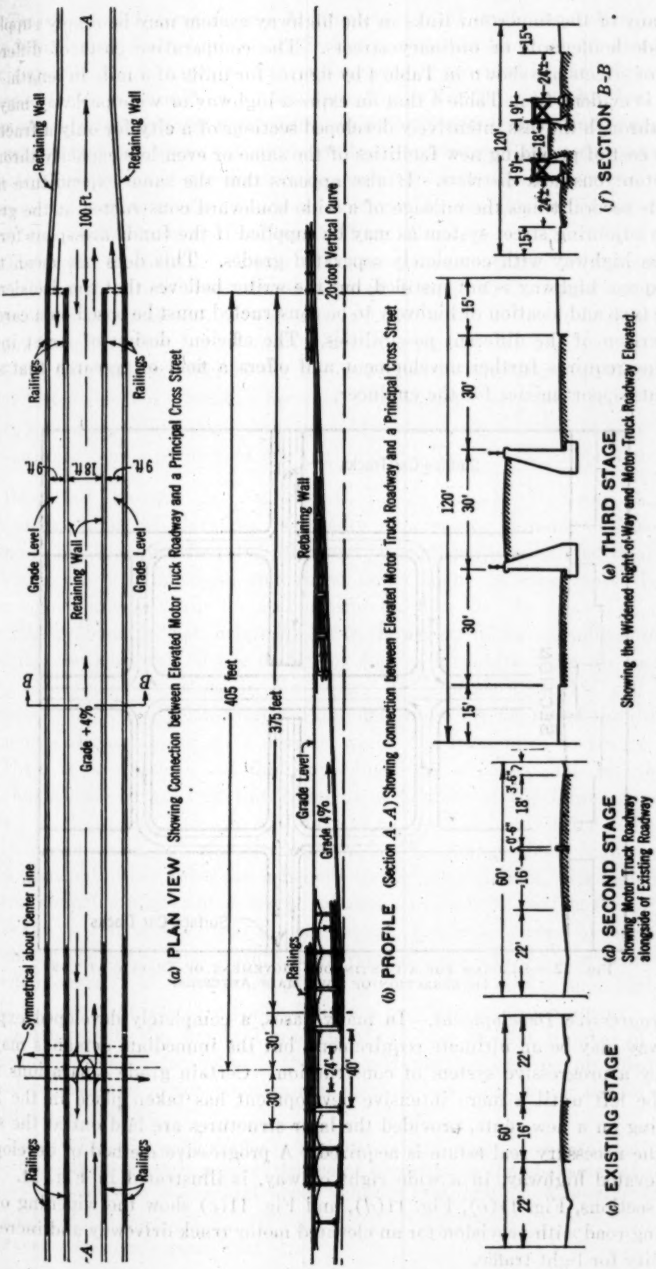

Symmetrical about Center Line
Railings
Railings
A
A
Grade +4%
B
B
100 Ft.
Grade Level
Grade Level
Retaining Wall
9 ft
18 ft
9 ft
Railings
Railings
100 Ft.
Retaining Wall
Retaining Wall
(a) PLAN VIEW Showing Connection between Elevated Motor Truck Roadway and a Principal Cross Street
30'
24'
30'
40'
405 feet
375 feet
Railings
Grade 4%
Grade Level
Retaining Wall
20-foot Vertical Curve
(b) PROFILE (Section A-A) Showing Connection between Elevated Motor Truck Roadway and a Principal Cross Street
22'
16'
60'
(c) EXISTING STAGE
22'
16'
60'
0'-6"
18'
3'-6"
(d) SECOND STAGE
Showing Motor Truck Roadway alongside of Existing Roadway
15'
30'
120'
30'
30'
15'
(e) THIRD STAGE
Showing the Widened Right-of-Way and Motor Truck Roadway Elevated.
15'
24'
9'
18'
9'
120'
24'
15'
(f) SECTION B-B

FIG. 11.—SKETCHES SHOWING POSSIBLE STAGES OF DEVELOPMENT OF MOTOR-TRUCK HIGHWAYS.

Many of the important links in the highway system may be amply supplied by wide boulevards or ordinary streets. The comparative costs of different types of streets are shown in Table 4 by figures for units of a mile in length.

It is evident from Table 4 that an express highway or wide parkway may be built through the less intensively developed sections of a city for only a fraction of the cost of providing new facilities of the same or even less capacity through important business districts. It also appears that the same expenditure may provide several times the mileage of a wide boulevard constructed at the grade of the adjoining street system as may be supplied if the funds are spent for an express highway with completely separated grades. This does not mean that the express highway is not justified, but the writer believes that the decision as to the type and location of highway to be constructed must be based on a careful comparison of the different possibilities. The efficient design of street intersections requires further development and offers a field of research that still presents opportunities for the engineer.

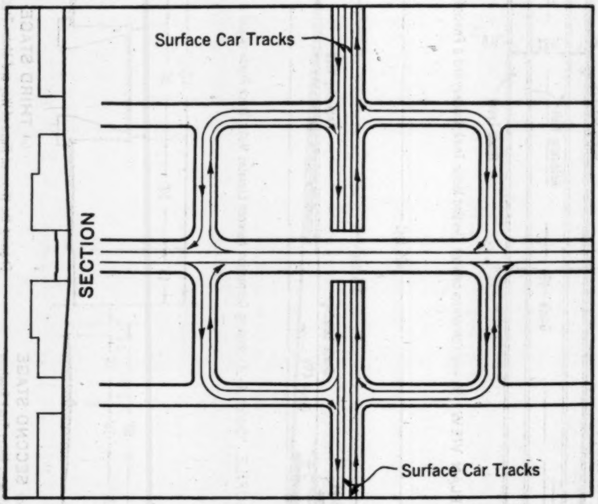


FIG. 12.—DIAGRAM FOR A CONTINUOUS MOVEMENT OF TRAFFIC AT THE INTERSECTION OF TWO MAIN ARTERIES.

Progressive Development.—In many cases, a completely developed express highway may be an ultimate requirement, but the immediate demands may be met by a progressive system of construction. Certain grade separations may well be left until a more intensive development has taken place in the land fronting on a new route, provided the later structures are laid out at the start and the necessary real estate is acquired. A progressive method of developing an elevated highway, in a wide right of way, is illustrated in Fig. 11. The road sections, Fig. 11(*c*), Fig. 11(*d*), and Fig. 11(*e*) show the widening of an existing road with provision for an elevated motor-truck driveway and increased capacity for light traffic.

Treatment of Intersections.—A complete type of grade separation between two intersecting main highways, offering unobstructed movement along each route and for any turning traffic between them, is provided by the "clover-leaf" intersection as used on New Jersey State Route No. 25, at Woodbridge. This, however, is extravagant of space except in undeveloped country and cannot well be used in business areas. A single grade separation and the use of adjoining streets, however, may approximate the same results in a built-up area. A suggestion for such a solution has been developed by the Regional Plan of New York and Its Environs and is shown in Fig. 12.

The traffic circle is a solution of intersections that may be the only one feasible where several radiating routes of comparatively equal importance intersect. For an intersection of only two routes the writer feels that it will seldom be satisfactory. The substitution of a rectangular area for the circle, with the long axis parallel to the major of two intersecting highways, offers a far better solution. Studies for such intersections have been made by Mr. Fritz Malcher.[15] They might well provide a temporary solution at some intersection points and a permanent one at points of only secondary importance.

DONALD M. BAKER,[16] **M. AM. SOC. C. E.** (by letter).—Highway planning, like other phases of planning, is rapidly approaching a more or less exact science. The economic features, while not yet sufficiently evaluated to apply to all cases, are still adequate enough to make such planning more than an approximate guess. While the author himself states that the method of attacking this problem is not original, he is, however, to be complimented for applying the approach to the design of such a structure from an economic point of view, as described in his paper. It is hoped that this will be an incentive for more experimental work to determine the values of the different elements entering into such a project, which can be applied to future cases.

There is no question but that super-highways, such as that described by Mr. Lavis, will be built in the future in a number of the larger American cities. Each project, however, will have to be developed only after a thorough study of economics, and also of the method of financing. Such highways, far more than any other type, are a benefit to the users thereof, although in the general financing plan care should be taken not to overlook the fact that, if these highways do relieve the local street systems, residents served by such local streets may be saved large future improvement costs and could reasonably be expected to contribute some small percentage toward the cost of such highways.

It is noted that only five traffic lanes are provided on this highway. While this is contrary to the standard practice of creating an even number of lanes, nevertheless, the writer believes that an odd number can be provided with considerable saving in construction and right-of-way costs, under the following conditions of traffic:

1.—Where the peak traffic in both directions is not of equal density.

2.—Where there is a great degree of diversity in the character of traffic, such as many high-speed vehicles and many slow-moving trucks.

---

[15] *The American City*, September and October, 1929, and August, 1930.

[16] Cons. Engr., Los Angeles, Calif.

In the first condition traffic studies often show that during morning or evening peaks traffic in one direction will amount to from 150 to 200% of that in the other direction. In this case the odd-numbered lane can be used to accommodate traffic moving in the direction of heaviest flow. If a reversal of this occurs at another time of day, the odd-numbered lane may be used for the traffic in the other direction. A little education is sufficient to adjust the public to this situation, and under these conditions an even number of lanes may normally represent useless expenditure.

Under the second condition in which, say, three or five lanes are provided (particularly three lanes), heavy slow-moving vehicles must be passed by faster and lighter vehicles and the existence of an extra lane used only for such passage will greatly expedite the traffic.

This all points to the fact that, before right-of-way and paving widths are definitely determined, a careful study of existing and prospective traffic should be made, in order to find out whether the extra lane necessary to furnish an odd number of trafficways is, or will be, necessary.

EDWIN F. WENDT,[17] M. AM. SOC. C. E. (by letter).—This paper is a notable contribution to the science of engineering in its relation to transportation. When the late Arthur M. Wellington, M. Am. Soc. C. E., wrote his book entitled "Economic Theory of Railway Location" fifty years ago, the Engineering Profession did not foresee that in 1930 the same principles would be applied to highway design by reason of the growth of traffic due to the extensive use of motor cars.

The author calls attention to the fact that, until recent times, highways were the developments of old trails made by man before the era of wheeled vehicles. The Hon. Henry W. Temple, Representative in Congress of the 25th Pennsylvania District, elaborated this same fact in an address delivered before the Service Clubs of Washington, Pa., September 25, 1930, on the subject "Some Historical Associations of the National Pike". This address contains much interesting information for the engineer.

Mr. Lavis very properly points out that engineering and economics are indissolubly linked together, and this applies to highways as well as railways. He has demonstrated by his thorough analysis of the problem of highway design that the true engineer is an economist, and that economics is essentially a part of engineering. The rise, the development, and the phenomenal growth in the number of motor cars require the design and construction of highways in accordance with scientific principles, so that there will be a true economic proportion between costs of such construction and the cost of operation of the motor vehicles which use the highways. The author is a pioneer in the application of the principles of the economic theory of railway location to the design, construction, and operation of modern highways.

H. W. HUDSON,[18] M. AM. SOC. C. E. (by letter).—The author has referred to a change made in the alignment and profile of Route 25 between the Pennsylvania Railroad crossing in Jersey City and a point west of the Passaic

---

[17] Cons. Engr., Washington, D. C.

[18] Engr. in Chg. of Constr., State Highway Comm., Jersey City, N. J.

River in Newark, which change was made subsequent to his connection with the project.

The permit issued by the War Department for the construction of the movable bridges having 35-ft. normal clearance, which had been adopted for the crossings of the Hackensack and Passaic Rivers, stipulated that the existing bascule bridge crossing the Hackensack River on Lincoln Highway should be removed following the completion of Route 25. The permit also stipulated that the Passaic River Bridge should be raised to provide a minimum clearance of 35 ft.

Realizing the impracticability of closing Lincoln Highway, due to the removal of the Hackensack River Bridge, as called for by the War Department's permit, the Highway Commission decided to construct a tunnel to take the place of the bridge. This decision led to considerable adverse criticism by interests more or less directly affected. It was appreciated, however, that the cost of this tunnel, as well as the reconstruction of the Passaic River Bridge, required by the War Department permit, would add so materially to the total expenditure that, under these conditions, the adopted plan, involving movable bridges, might not remain the most economical. Further studies, therefore, were made of a proposal involving tunnels under both rivers and an open cut below the meadow level across the Kearny Peninsula, as well as a plan involving bridges with sufficient under-clearance to permit the use of fixed spans.

Either of these proposals would not affect the river traffic, and it was assumed, therefore, that the War Department would not require any changes in the Lincoln Highway bridges in connection with issuing a permit for either of these types of river crossings, and hence no consideration was given to this contingency in preparing the comparative estimates. In the former, it was necessary to consider the possibility of ultimately having to construct a roof over this cut coincident with the development of the contiguous property. In such event, it was realized that the problem of ventilation would be a serious element.

The alignment for both the tunnel and high-level viaduct plans was identical and was found to be several hundred feet shorter than that of the adopted location which contemplated movable bridges with a normal clearance of 35 ft. Either proposal involved a saving in curvature, but increased the rise and fall over the 35-ft. clearance scheme.

The construction and right-of-way costs of the route involving movable bridges (including the raising and reconstruction of the Passaic River Bridge and the construction of tunnels under the Hackensack River, both on Lincoln Highway), was found to be greater than the high-level viaduct route. Similarly, it was found that the construction costs of tunnels under the rivers, with the open depressed cut through the meadows, would be in excess of either the 35-ft. plan, or the high-level viaduct. Investigation of automobile operating costs of the three proposals also indicated the high-level viaduct project to be the least costly.

The Highway Commission, therefore, rescinded its action as regards the 35-ft. level bridge location and adopted the 135-ft. high-level viaduct. The latter will provide a highway entirely free from any interruptions to its traffic caused by water-borne traffic.

Another change made subsequent to the author's connection with the work was that of the three crossings of the Elizabeth River, in Elizabeth, N. J. As the waterway at this point is under the jurisdiction of the War Department, the permit for the crossings called for the construction of movable bridges. Authority was given to erect these structures as fixed spans until such time as it might be necessary to provide for navigation, when the operating mechanism would have to be installed.

The construction through Elizabeth provided for all streets to intersect or cross the highway at grade. It was evident that a serious traffic hazard would obtain at the crossing of Elizabeth Avenue which is the principal east and west thoroughfare. It was finally arranged to carry the highway over this street on a bridge, and to continue it on an elevated structure southward to, and including, the three river crossings where fixed span bridges with a vertical clearance 22 ft. above mean high water were used. The War Department waived the requirement of movable bridges and approved of the construction of the fixed-span bridges with the clearance mentioned. The wisdom of this viaduct construction has been fully justified.

Fred Lavis, M. Am. Soc. C. E. (by letter).—The writer appreciates the generally favorable comment on his paper, especially because of the prominence of many of the commentators in the field of modern highway construction along the lines of development which take into due consideration the effects of such highways and the details of their location on the economic operation of the vehicles using them.

The commentators recognize also the nice balance which must be preserved between the bare application of any economic theory in a problem of this kind and the "other" factors which affect the location and design. It is the ability to strike this balance accurately, giving proper weight to economic as well as technical considerations, which marks the true engineer.

The discussion by Mr. Lewis is particularly interesting, as he gives a complete outline of the various types of construction which have been developed thus far in the endeavor of engineers to find solutions of these problems to fit the needs of different localities and conditions.

Mr. Downer also refers only too briefly to the development of the highways of the Westchester County Park System. This project originated primarily as a park development along the Bronx River Valley between the northerly end of New York City and Kensico Reservoir and was the practical realization of a dream for the preservation of scenic beauty, with a highway as an incidental feature. Before the Bronx River Parkway was completed, however, the highway feature had grown to be a most important factor, and fortunately it was realized by Mr. Downer at an early stage in the development that special attention would have to be paid to its design as an "element in transportation".

In the later improvements through the other three valleys running northerly from New York City the landscape features of the parkways have been developed, as adjuncts of the highways, and the design of these highways has been more and more influenced in these later years by traffic requirements. Mr. Downer and the Westchester County Park Commission deserve the highest congratulations for having achieved that equitable balance between traffic needs and the artistic and other requirements which is so eminently the earmark of true engineering.

It will be realized, however, that the location of Route 25, in New Jersey, through an entirely urban and industrial and manufacturing district, did not permit this type of development, and this all serves to point out the need of the individual study of each problem by properly qualified engineers in order to find the solution best adapted to the needs of each particular case.

Perhaps a word may be permitted in regard to highway widths. On Route 25, New Jersey, the Elevated West Side Highway in New York, and others of this type, no provision is made for other than moving traffic, in which case the writer believes an odd number of lanes (five of 10 ft. each in the case of Route 25) is indicated. The principal reasons for this are pointed out in the discussion by Mr. Baker.

There seems, however, to be a general tendency to establish State Highway widths for main routes at 20 ft., or 40 ft. (two or four lanes). With evenly balanced traffic, and the need of giving some consideration to parking requirements, this may be correct; but the traffic in the vicinity of big cities, especially big terminal cities like New York, is not balanced and, as Mr. Baker points out, the extra lane is justified. This is also more particularly the case when part of the traffic is commercial and is especially noticeable on some roads leading to New York on Sunday afternoons and evenings, with many vehicles carrying produce for the markets at the time that the peak load of tourist traffic is returning to the city.

In regard to Route 25, New Jersey, it was estimated that four full lanes of moving traffic would be required to take care of peak loads, and that it was only ordinary prudence to provide an extra lane for emergencies and passing to keep four full lanes continuously moving.

The discussion by Colonel Hudson, outlining the changes in design on two sections of Route 25, is a welcome contribution and serves to complete the record of the project. Referring to the map and profile, Figs. 1 and 2, the section first referred to is that between Stations 180 and 330. For the four years (1925 to 1929) during which this matter was studied by W. G. Sloan, M. Am. Soc. C. E., State Highway Engineer, and the writer, there were long drawn-out controversies with the municipal authorities and others as to the relative merits of a line on this general location, with crossings of the Hackensack and Passaic Rivers by tunnels or with movable bridges with 40 ft. clear head-room. The writer and Major Sloan were opposed to a tunnel line with the connecting low-level trenches for this distance of nearly 3 miles.

The writer is entirely in accord with the final decision to build this section of the highway as a viaduct with 135 ft. clear head-room over the rivers so long as the War Department and the New Jersey River and Harbor Board insisted on this as the minimum clearance for fixed spans. It may be pointed out, however, that if such a structure had been proposed in the first place, it probably would have imperiled the entire project, partly because of the expense involved, but largely because of the general impression which would have been created in the minds of the public that it was too grandiose.

In passing also it may be noted that, in the writer's opinion, neither the War Department nor the Board of Commerce and Navigation of New Jersey has an adequate appreciation of the relative or economic value of terrestrial as compared with the water-borne traffic of this area, and, consequently, the taxpayers of the State are unduly penalized, not only in expenditures for unnecessarily costly structures, but in the costs of traffic delays.

In regard to the change at Elizabeth, the project as finally carried out, except for a slightly higher elevation over the Elizabeth River, is almost exactly that recommended by the writer and adopted by the Highway Commission early in 1926. It was strenuously opposed, however, at that time, by almost all the interests in Elizabeth. Many and various proposals for other routes were studied and suggested during the next three years. Agreements were reached only to be thrown over at the last moment, and Colonel Hudson is entitled to a great deal of credit for finally persuading "the powers that be" to agree to the project as finally constructed, which the writer understands is now considered a most acceptable solution of the problem.

The original report of the Advisory Board of the Highway Commission, of which Major Sloan was Chairman, showed an estimated cost of approximately $20 000 000 for the entire 13 miles from the Holland Tunnel to Elizabeth, with bridges over the Hackensack and Passaic Rivers (for tunnels, $28 000 000), and the writer has always felt that the Commission was entitled to the very highest commendation for the splendid manner in which it upheld the report of its engineer and decided to go ahead with a project of this magnitude with little—if any—precedent to guide it.

After the work was finally authorized, more detailed studies showed necessary or desirable developments which brought the estimated cost of the project, as finally worked out under the writer's direction, up to a total of approximately $35 000 000; and the final cost, with the change to the high level as referred to in Colonel Hudson's discussion, to approximately $40 000 000.

To the layman this may appear to be another instance of the inadequacy of engineers' estimates, but consideration should be given to two most important factors. First, the very great number of different interests which had to be served or satisfied. Above all, there was the State, then the counties, the municipalities, the chambers of commerce, the railroads, the public utilities, the navigable waterways interests, individual manufacturing and commercial interests, and, finally, the private citizens, all of which had a voice in determining some detail of the project. The very important change in Newark shown in Fig. 7, which added more than $1 000 000 to the cost of the highway, is a good illustration. Second, this was a pioneer project of its kind. Very few people were able to visualize it or what the plans of the Highway Commission contemplated. In the writer's opinion this perhaps was fortunate, because if its magnitude had been realized it is doubtful whether it would have been carried out.

Finally, the writer feels that, not only is Major Sloan entitled to the highest credit for the original conception of the project as the only reasonable solution of the problem of highway transportation through this area, but in even greater degree for his patience and persistence in insisting that it be carried through on the basis of true engineering principles.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1784

SILTING AND LIFE OF SOUTHWESTERN RESERVOIRS[1]

By R. G. Hemphill,[2] Assoc. M. Am. Soc. C. E.

With Discussion by T. U. Taylor, M. Am. Soc. C. E.

Synopsis

This paper cites records of the silt load of several of the principal streams of the Southwest to indicate the importance of the silt problem in that region and the necessity for a comprehensive study of it to meet adequately water storage problems which will arise in the future. There follow descriptions of methods used, records obtained, and some tentative conclusions drawn from an investigation of silt in Texas streams, which is now (1929) entering its sixth year.

Erosion is governed by many factors, the more important of which are the amount and intensity of rainfall, and the character of topography, surface cover, and soil. The combination of these factors in the Southwest is favorable in a marked degree to heavy erosion.

Rainfall varies from an annual mean of about 30 in. at the eastern border of the semi-arid area of Texas to an annual mean of less than 5 in. in sections of Arizona and California.[3] Light rains are fairly numerous, but constitute only a small percentage of the total precipitation. As far as erosion is concerned, the significant feature of the precipitation is the large percentage of the total which occurs as torrential rains. The U. S. Weather Bureau reports 23.11 in. in 24 hours in September, 1921, at Taylor, Tex., the highest precipitation ever officially recorded in the United States for that period,[4] and apparently reliable unofficial records for the same storm at points near-by are higher. Intensity rates of 4 to 6 in. per hour are not uncommon in Texas, and rates for other States of the Southwest are comparable.

---

[1] Presented at the meeting of the Irrigation Division, Dallas, Tex., April 25, 1929.

[2] Irrig. Engr., U. S. Dept. of Agriculture, San Antonio, Tex. Mr. Hemphill died on June 19, 1930.

[3] "Precipitation and Humidity," by J. B. Kincer, Section A, Pt. II, Atlas of American Agriculture, U. S. Dept. of Agriculture.

[4] *Monthly Weather Review*, September, 1921, U. S. Weather Bureau.

Such rainstorms produce a rapid and heavy surface run-off which is very effective in surface erosion; and the quick collection of the run-off in drainage channels produces sharp floods with high velocities which continue the processes of erosion and transportation of silt. As might be expected, the flow of the streams of the region is generally characterized by long periods of low stage and short, sharp floods of relatively great volume. The Colorado River and Rio Grande, which carry spring floods from melting snow in their upper basins, must be noted as exceptions, although in the case of the Rio Grande much of the spring flood is diverted in the San Luis Valley of Colorado.

Except areas in Texas, the topography of the region is predominantly mountainous with steep slopes permitting a rapid run-off. The State of Texas includes a relatively small area of smooth, gently sloping plains and a much larger area of rolling plains with slopes sufficiently heavy to produce rapid run-off.

In general, the surface cover[5] of the region may be classified as sparse. Forests are limited to Central Texas where there are small scattered areas of oak and hickory and to the higher altitudes of Arizona and New Mexico where piñon, juniper, yellow pine, and fir are found. Creosote bush covers a large area in the four States and some sage-brush is also found. The remaining areas are covered by short, tall, or mesquite grass. The effectiveness of the grass cover in checking run-off and erosion has been reduced to some extent by excessive grazing.

The region contains mostly finely divided alluvial soils, in many instances extending to great depths. The tendency to erode is relatively high. While the area of land under cultivation comprises a small part of the total area, doubtless, under present conditions, such land contributes substantially to the silt yield of the region.

QUANTITY AND EFFECT OF SILT IN STREAMS

To the most casual observer it is apparent that the factors noted are effective in loading the streams of the region with silt. The problems involved were recognized at an early date. Since about 1890, a number of agencies, including the U. S. Geological Survey, the U. S. Bureau of Reclamation, the U. S. Department of Agriculture, the various State experiment stations, and canal managements, have investigated the bearing of silt on some particular existing or proposed project. To cite the results of only a few of these investigations will indicate the extent and character of the problem involved.

Colorado River.—The silt load of the Colorado is of particular interest in view of the construction of an immense reservoir in its Lower Canyon Section above the Hoover Dam. Records for 18 years at Yuma, Ariz., show an average annual load of suspended silt of 183 759 000 tons,[6] or, approximately, 0.84% by weight of the average annual discharge of 16 200 000 acre-ft. Assuming a drainage area of 225 000 sq. miles above Yuma, the average yield of silt per square mile per year was 817 tons.

---

[5] "Natural Vegetation," by H. L. Shantz and Raphael Zon, Section E, Pt. I, Atlas of American Agriculture, U. S. Dept. of Agriculture.

[6] "Silt in the Colorado River and Its Relation to Irrigation," by Samuel Fortier, M. Am. Soc. C. E., and Harry F. Blaney, Assoc. M. Am. Soc. C. E., U. S. Dept. of Agriculture, *Technical Bulletin No. 67*, 1928.

By comparing records for short periods at Topock and Yuma, Ariz., and on Imperial Valley canals, and assuming that 1 cu. ft. of deposit would contain 85 lb. of dry silt, Fortier and Blaney[6] reached the conclusion that the total load of bed and suspended silt at the lower end of the Canyon Section of the river averages 137 000 acre-ft. per year. For the drainage area of 171 000 sq. miles above Topock, this is equivalent to a silt yield of 0.80 acre-ft. per sq. mile per year.

If this estimate holds good, the capacity of a reservoir formed by a 500-ft. dam[7] at either the Boulder Canyon site or the Black Canyon site will be reduced by one-half in less than 75 years by deposits of silt.

Gila River.—Only a few short-term records of silt in the Gila are available to confirm its local reputation of being the muddiest stream in the United States. A Board of Army Engineers in reporting on the San Carlos Project in 1914[8] estimated an average of 1.3% of silt by volume based on 70 lb. of dry silt in 1 cu. ft. of deposit. The equivalent by weight is 1.46 per cent.

Several silt surveys have been made in the Roosevelt Reservoir on Salt River, a tributary carrying less silt than the Gila. In 1925, Cragin[9] reported an accumulation of 101 000 acre-ft. of deposit after 20 years of service. At the average rate of silting shown by the surveys, about 160 years will be required for deposits to reduce the capacity of the reservoir by one-half.

Rio Grande.—On the Rio Grande, silt records are available for long periods at El Paso, Tex., and San Marcial, N. Mex. Follett, in his report on silt in this stream,[10] combined the records for these two stations and concluded that at San Marcial, for the period from 1897 to 1912, inclusive, the annual discharge averaged 1 192 000 acre-ft., and contained suspended silt to the amount of 1.41% by weight. Relying on one sample taken under carefully selected conditions, he assumed that 1 cu. ft. of deposit of this silt in a reservoir would contain 53 lb. of dry material. Converting the weight on this basis, he arrived at an estimate of 19 739 acre-ft. of suspended silt per year at San Marcial.

That Follett used good judgment in selecting his volume-weight factor seems to be borne out by the silting of the Elephant Butte Reservoir immediately below San Marcial. Surveys made by engineers of the U. S. Bureau of Reclamation indicate that silt deposits are filling the reservoir at the rate of approximately 20 000 acre-ft. per year.[11] If this proves to be an average rate, the original capacity of 2 638 860 acre-ft. will be reduced by one-half at the end of 61 years of service.

---

[7] "Water Power and Flood Control of Colorado River below Green River, Utah," by E. C. LaRue, M. Am. Soc. C. E., Capacity Tables in U. S. Geological Survey *Water Supply Paper No. 556.*

[8] "Amount of Silt That Would Be Deposited in Reservoir at San Carlos," by D. S. Hughes, M. Am. Soc. C. E., House Doc. 791, 63d Cong., 2d Sess., San Carlos Irrigation Project, Arizona, Appendix G.

[9] Unpublished report to U. S. Bureau of Reclamation, by C. C. Cragin, M. Am. Soc. C. E., Gen. Supt. and Chf. Engr., Salt River Valley Water Users Assoc.

[10] "Silt in the Rio Grande," by W. W. Follett, Dept. of State; International Boundary Comm.; Comm. for the Equitable Distribution of Waters of the Rio Grande.

[11] "Elephant Butte Dam," by P. I. Taylor, "Dams and Control Works Constructed by the Bureau of Reclamation," U. S. Dept. of the Interior.

Follett made no estimate of the silt rolled along the bottom of the stream, but stated his belief that it might possibly amount to as much as 25% of that suspended. Accepting this ratio as the maximum, the measured average annual deposit of 20 000 acre-ft. does not include more than 4 000 acre-ft. of bed silt. The remaining 16 000 acre-ft. of deposit is, therefore, left to be accounted for by the annual average of 22 785 000 tons of suspended silt found by Follett. The combination of this volume and weight of silt gives a unit value of 65 lb. of dry silt to 1 cu. ft. of deposit. It is probable that the 16-year record of suspended silt and the 10-year record of reservoir deposits did not develop true averages, but it is believed that they were accurate enough to indicate fairly that suspended silt in the Rio Grande will form deposits containing from 55 to 65 lb. per cu. ft. of dry silt.

Pecos River.—In 1904, a survey of Lake McMillan on the Pecos River near Carlsbad, N. Mex., showed that in ten years of service its capacity had been reduced 42% by silt deposits.[12] This was at the rate of about 1 225 acre-ft. per year. In 1916, a Board of Engineers of the Bureau of Reclamation, on the basis of 21 years of service, arrived at an estimate of an average inflow of 2 400 acre-ft. of silt.[13] This was 0.8% by volume of the estimated annual run-off.

Colorado River of Texas.—The silting of the lake on the Colorado River near Austin, Tex., was carefully observed by Taylor[14] from the time of the construction of the first dam. In less than 7 years of service prior to the failure of the dam, silt deposits reduced the capacity of the lake from 49 300 to 25 741 acre-ft., an average of 3 490 acre-ft. per year. At this average rate, deposits would have equalled the total capacity of the lake in 14 years. The drainage area above the dam has been estimated at 38 200 sq. miles, and the annual deposit of 3 490 acre-ft., therefore, was equivalent to a silt yield of 0.09 acre-ft. per sq. mile per year.

The silting of this reservoir is convincing evidence of the necessity for a reasonable relation between the size of a reservoir on a silt-laden stream and the run-off it receives. The mean annual discharge of the Colorado at the dam is 1 900 000 acre-ft., or more than thirty-eight times the original capacity of the reservoir. Since the entire silt load of the stream, during the first years of service at least, was probably retained in the reservoir, it is not surprising that silting proceeded at the rapid rate of 7% of the capacity per year. Had the reservoir been twenty times as large, for instance, and subject to the same run-off and silt yield, silting would have proceeded at a rate of less than 0.4% per year, and approximately 280 years would have been required for the accumulation of deposits equal to the capacity of the reservoir.

---

[12] "Water Resources of the Rio Grande Basin," by Robert Follansbee, M. Am. Soc. C. E., and H. J. Dean, U. S. Geological Survey, *Water Supply Paper No. 358.*

[13] "Silt Deposited in Reservoirs—Carlsbad Project," by T. E. Weymouth, D. C. Henny, R. F. Walter, Members, Am. Soc. C. E., and L. E. Foster, Esq.; unpublished report, Board of Engrs., U. S. Reclamation Service.

[14] "The Silting of the Lake at Austin," by T. U. Taylor, Univ. of Texas, *Bulletin No. 2439.*

Trinity River.—In 1928, Taylor made a survey of Lake Worth[15] and found that in 13 years of service 13 837 acre-ft. of silt had accumulated in the reservoir. The average annual deposit, therefore, was 1 065 acre-ft., and amounted to 2.3% of the original capacity of the lake. On this basis, about 45 years would be required for deposits to equal the original capacity of the lake.

The drainage area above the dam is 1 870 sq. miles. The annual deposit of 1 065 acre-ft., therefore, indicates an average annual silt yield of 0.57 acre-ft. per sq. mile.

NEED FOR INVESTIGATION

These examples clearly demonstrate the necessity of a careful consideration of the silt problem in connection with Southwestern reservoirs. While this region has lagged behind other sections of the country in the past, there is now evident a determination to bring about a full utilization of its water resources. In view of the erratic flow of the streams, it is certain that any considerable irrigation or power development must depend on stored water. It follows that one of the chief problems of the engineer of the future project will be to determine the silt load of the stream and to adjust the capacity of the reservoir to other requirements so as to give it the longest effective life within the range of feasible costs.

The best basis for an estimate of the silt load is a long-term record of silt measurements at the site of the reservoir. Such records have been available only in a very few instances, and when a new reservoir is proposed it is seldom practicable to delay the project several years while continuous records of the silt load are being obtained. The result is that estimates are based on short-term or fragmentary records at the site, or on records taken in another locality where conditions are assumed to be comparable.

Either procedure may lead to serious error. Under the circumstances, there can be no question of the real need for an extensive determination of the silt load of Southwestern streams, and the writer is optimistic enough to believe that such an investigation, taken in connection with soil erosion studies made by the U. S. Department of Agriculture and several State agricultural experiment stations, will uncover fundamentals of general application.

INVESTIGATION IN TEXAS

About 1923 the time seemed ripe for a general investigation of the water resources of Texas and of the means by which they might be brought to their maximum development. On the advice of the engineers and other forward-looking men of the State, the Legislature made a substantial appropriation and delegated to the State Board of Water Engineers the duty of carrying out a great part of the program. In addition to increasing the number of stream-gauging stations, and locating and surveying reservoir sites, it was considered necessary to determine the quantity of silt carried by the streams of the State. This silt investigation was inaugurated in 1924 by the State

---

[15] "Silting of the Lake at Austin, Texas," by T. U. Taylor, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1681.

Board in co-operation with the Division of Agricultural Engineering of the U. S. Department of Agriculture, with the writer in charge.

Due to limited funds, it was not possible to undertake the determination of the silt load of every stream of the State which offered suitable sites for reservoirs. The Brazos, therefore, was selected as the single basin which best represented in its various divisions the conditions found in other drainage areas. Sampling stations were established at nine gauging stations which were either near reservoir sites or below areas comparable to other drainage basins. Routine daily sampling was begun immediately. These records determined the silt load at any point in the Brazos Drainage Basin and afforded a reasonably sound basis for estimates of the loads of other streams of the State. As a check on such estimates, records were obtained at single stations established on the Colorado, San Antonio, and Nueces Rivers.

Sampling Device.—Samples were taken in 8-oz., narrow-mouthed, round, glass bottles. The bottle was placed in a holder on a hanger to which was attached a stream-line current-meter weight varying in size from 7 to 100 lb., according to the velocity of the water to be sampled. Wired to the hanger was a coil spring to which was attached a rubber stopper and a cord by which the stopper might be pulled. The chief purpose of the spring was to counteract the force of the current on the cord and prevent the premature pulling of the stopper. The hanger was suspended by a cable, and, if the current was swift enough to require a heavy weight, a windlass fastened to an A-frame was used to raise and lower it.

Method of Sampling.—The difficulty and cost of securing samples and handling them in the laboratory made it necessary to determine the minimum number which could, when averaged, be relied upon to represent accurately the average silt content of the entire cross-section of the stream. To arrive at the desired result, a large number of samples were taken throughout this cross-section for different stages at several stations, the number for each sampling ranging from 30 to 60. In each vertical, samples were taken at every one-tenth or one-fifth of the depth, at the surface, and as close to the bottom as it was possible to go without disturbing the condition of flow to an extent which would have vitiated the result. From these samples, vertical curves of silt percentage were plotted. Thus, it was determined graphically that, within the limits of permissible error, a sample taken at six-tenths of the depth would give the average for the vertical.

In a similar manner the horizontal curve of silt percentage was plotted for each stage at each station, and it was found that the average of percentages at the center of the stream and at distances of one-sixth of the width from each bank would give the average percentage for the cross-section. For routine sampling, therefore, samples were taken at six-tenths of the depth, and at one-sixth, three-sixths, and five-sixths of the width of the stream. The probable error in this method is well within the limit fixed by the degree of accuracy which can be obtained in stream gaugings at stations such as are ordinarily found in Texas.

Laboratory Methods.—The three daily samples from each station were forwarded to Austin where the silt content was determined in a laboratory maintained for that purpose. The method used involved filtering out the silt, drying it, and obtaining its dry weight. The routine required the weighing of: (1) A dried filter paper; (2) the bottle and sample; (3) the empty bottle; and (4) the dried filter paper and silt. The net weight of the sample and the net weight of the silt, obtained from these weights, gave the percentage.

In order to retain the finest silt, a tight, stout filter paper was used. Drying was done in an automatic electric oven set to maintain a temperature of 110° cent. To overcome troublesome variations in atmospheric moisture, the filter papers, before being used, were dried out at 110° cent., cooled in a desiccator, and taken out one or two at a time and weighed. By following the same procedure with the filter paper containing silt, after drying was completed, the error occasioned by the rapid absorption of atmospheric moisture was reduced to a negligible point.

When the samples showed a large quantity of silt, one of each set of three was first poured into a 40-in. glass tube with a uniform $\frac{3}{4}$-in. bore, and allowed to settle for one week to determine the percentage by volume for that period of settling. The sample was then passed through the regular routine to determine the percentage by weight. This gave for the same sample the percentage by weight and the percentage by volume after a week of settling.

It is recognized that volumetric percentages determined by seven days of settling in a glass tube do not directly represent the volume the silt will occupy when deposited in a reservoir. It is possible, however, that a proper reduction factor can be found and applied with little, if any, greater chance for gross error than exists in the selection of a factor for converting a given weight of silt to the volume it will occupy when deposited in a reservoir.

Suspended Silt in Brazos River.—To indicate the character of the results obtained, tentative data covering the load of suspended silt at four of the Brazos Basin Stations during the first three years of the investigation, are given in Table 1. Three of the stations included are on the main stream: (*a*) Mineral Wells, which is in the upper part of the basin and catches the combined flow of the principal upper tributaries; (*b*) Waco, near the middle of the basin immediately below the junction of the Bosque River; and (*c*) Rosenberg, only a short distance above the mouth of the stream. The station near Aspermont, on the Double Mountain Fork, is included as representing heavily laden branches of the stream near its source. The results involve the silt load for each day, the percentage shown by the three samples being applied to the discharge in order to determine the daily silt tonnage. The average percentages shown are based on the total tonnage and the total discharge for the period.

As was to be expected, the highest average silt content—2.13%—was found at the Aspermont Station on the Double Mountain Fork, which drains an area particularly subject to erosion. At Mineral Wells, the flow includes the discharge of other upper tributaries carrying relatively less silt and the percentage drops to 1.01. Below Mineral Wells, increasing rainfall dilutes the silt content and lowers the percentage to 0.86 at Waco and to 0.43 at Rosenberg. The percentages for 1925 were uniformly much higher than for the other two years. This is accounted for by the fact that 1925 was a year of unprecedented drouth, during which only two floods occurred, one in May and one in September. These floods, following dry periods of more than three months, carried extraordinarily high percentages of silt. For this reason it is believed that average percentages shown by records over a long period will be appreciably less than those shown by the three-year period.

TABLE 1.—SUSPENDED SILT SHOWN BY PRELIMINARY COMPUTATION OF RECORDS AT FOUR STATIONS IN THE BRAZOS BASIN FOR THE THREE-YEAR PERIOD FROM OCTOBER 1, 1924, TO SEPTEMBER 30, 1927.

Station.	Drainage area, in square miles.	Annual discharge, in acre-feet.	ANNUAL LOAD OF SUSPENDED SILT.		Annual yield per square mile, drainage, in tons.
			Tons.	Percentage.	
Double Mountain Fork near Aspermont, Tex.	7 980	170 000	4 942 000	2.13	619
Brazos, near Mineral Wells, Tex.	23 100	981 000	13 506 000	1.01	585
Brazos, at Waco, Tex.	28 500	1 629 000	19 054 000	0.86	669
Brazos, at Rosenberg, Tex.	44 000	5 188 000	30 528 000	0.43	694

It is interesting to note the rather close agreement in the average yield of silt per square mile of drainage above the four stations. The lowest average—585 tons for the area above Mineral Wells—is more than 84% of the highest average of 694 tons for the area above Rosenberg. By segregating the different areas, however, it is found that for the area which drains into the river between Mineral Wells and Waco the average yield of silt was 1 027 tons per sq. mile; and for the area between Waco and Rosenberg, 740 tons per sq. mile. The unusually high yield between Mineral Wells and Waco is believed to have been due partly to sharp floods in the Bosque River, but the general increase of yield down stream is probably due chiefly to increasing rainfall and increasing acreage under cultivation.

Solely for the sake of comparison, the volume of the silt load at each of the stations has been computed on the basis of 70 lb. of silt to 1 cu. ft. of deposit. At this rate, the suspended silt at Aspermont amounted to, roughly, 3 000 acre-ft., or 0.41 acre-ft. per sq. mile of drainage; at Mineral Wells, 9 000 acre-ft., or 0.38 acre-ft. per sq. mile; at Waco, 12 000 acre-ft., or 0.44 acre-ft. per sq. mile; and, at Rosenberg, 20 000 acre-ft., or 0.45 acre-ft. per sq. mile.

Reservoir Surveys.—An important phase of the investigation in Texas has been the actual measurement of deposits of silt in reservoirs. It was the intention to find a reservoir where conditions were such that the silt discharge of the stream could be measured for a period of years as it entered the reservoir and where at suitable intervals measurements could be made of the actual deposit in the reservoir, thus establishing for one set of conditions an absolute relation between measured silt in suspension and the corresponding deposit in the reservoir. Unfortunately, the required conditions could not be found and thus far (1929) this phase of the work has been confined to surveys of the actual deposits in Medina Lake, near San Antonio, Tex., Lake Worth, near Fort Worth, Tex., and Lake Kemp, near Wichita Falls, Tex.

The survey of Medina Lake showed that in 13 years of service silt deposits to the amount of 2 692 acre-ft. had accumulated. The yearly average was 207 acre-ft., which was equivalent to a yield of 0.35 acre-ft. per sq. mile of the approximately 600 sq. miles of drainage area above the dam. The capacity of the reservoir is 254 000 acre-ft. and, at the rate shown by the survey, deposits will equal the capacity in 1 225 years.

The survey of Lake Worth was made in 1925 and indicated that silting was taking place at the rate of approximately 1 000 acre-ft. per year. At this rate, deposits would equal the capacity of the reservoir in 47 years. This result was checked very closely by Taylor's survey made in 1928. The yearly average deposit of 1 000 acre-ft. is equivalent to a yield of 0.54 acre-ft. per sq. mile of drainage area above the dam. It is interesting to note that, if the average silt yield, in tons per square mile, from the area above this reservoir is comparable to the yield from the near-by Brazos drainage, as shown by records at Glenrose, the deposit in the reservoir contains less than 60 lb. of silt per cu. ft.

A complete survey of deposits in Lake Kemp was attempted in its second year of service; but brush left in the basin was so heavy in certain sections that lines could not be put through. It was not considered advisable to make an estimate of the total silt deposit based partly on such random soundings as could be made in these sections. The maximum depth of silt, 4 ft., was found at the upper end of the lake, but fine silt to a depth of not less than 2.5 ft. was found in the old river channel throughout the reservoir.

Characteristic delta formations, largely of coarse material, were found at the head of these lakes. Below the delta areas, the deposits consisted of fine material and were distributed so as to lead to the belief that the finest silt when deposited will flow along the bottom of the reservoir until blocked by the dam. There seems to be no other explanation for the fact that the thickness of the layer of fine material in the case of each reservoir was little less at the dam than it was 8 or 10 miles above. If this condition proves to be general it indicates a possibility of relieving the reservoir of some silt by making a practice of drawing off water through the lowest outlet gate.

The Weight of Deposited Silt

The most satisfactory and, in fact, the only practical method of determining the amount of silt in samples of river water is by weight; but since the space the silt will occupy in a reservoir is the important consideration, it is necessary to fix a relation between the weight of the silt and its volume when deposited. There is a wide difference of opinion in regard to this relation, and values have ranged from 120 lb. per cu. ft., used by Humphreys and Abbot[16] in estimating the volume of silt carried in suspension by the Mississippi River, to 53 lb. per cu. ft., used by Follett in connection with the Elephant Butte Reservoir. It seems clear that the volume-weight factor of a deposit will depend chiefly on the proportions of fine and coarse material, the degree of separation which takes place on deposit, the depth of deposit, and whether it is subject to exposure and consequent drying, consolidation, and mixing by the scouring action of later inflow. The selection of a factor for a given reservoir should be governed by a consideration of all these elements.

---

[16] Report upon the Physics and Hydraulics of the Mississippi River, by A. G. Humphreys and H. L. Abbot, Bureau of Topographic Engrs., U. S. War Dept., 1861.

The proportions of the different sizes of silt carried by streams vary greatly, and under ordinary conditions there is a substantial separation into different grades when the material is deposited. Silt-laden water entering the quiet water of a reservoir tends to continue its course by pushing aside and upward the clear water of the reservoir, and its velocity is reduced more or less gradually. This action is responsible for the rather high degree of separation which has been noted in actual deposits in reservoirs. A fluctuating water level, drying deposits, and the scouring action of a fluctuating inflow are mainly responsible for such mixing of different sizes of silt as occurs.

Some writers have assumed that increasing hydrostatic pressure may be counted on to compact deposits, but there seems to be little ground for the assumption. The writer believes that, unless exposed, deposits will be compacted to a significant degree only by the weight of the silt itself and the overburden of new deposit. In the surveys of Texas reservoirs, no difference in density could be detected in bottom deposits of fine silt under depths of water ranging from 5 to 125 ft. Deposits of fine silt will compact very little unless exposed. Those in the lake at Austin, Tex., for instance, are so loose that a 1-in. rod can be forced into them with one hand to a depth of 10 to 12 ft. When exposed to air and sunlight, however, they dry out, shrink heavily, and crack, and on being submerged again they do not swell to their original volume.

As a guide in determining the relation of volume to weight under various conditions, a large number of sample cubes were taken from deposits in place in Texas reservoirs and the dry weight of the silt was determined. Exposed bars of coarse silt, such as is found at the head of reservoirs, averaged 92 lb. of dry material to 1 cu. ft. of deposit, while finer silt in much the same location averaged 82 lb. Samples from the surface of deposits near the middle of reservoirs averaged 55 lb. of dry material to each cubic foot. Samples of the finest material taken from submerged deposits in old river channels in reservoirs averaged 31 lb. of dry material to 1 cu. ft.

BED SILT

In addition to the silt carried in suspension, a certain quantity is moved along the bottom of the stream; but no investigator has yet devised a satisfactory method of measuring its amount. Estimates vary widely. Humphreys and Abbot, from a study of the movement of bars at the mouth of the Mississippi, estimated the bed load to be 11% of the suspended load by volume;[17] but in arriving at the volume of suspended silt, a volume-weight factor of 120 lb. per cu. ft. was used. It is now generally recognized that a much lower factor should be adopted, in which case the percentage of bed silt would be substantially lower. By the use of traps in the San Carlos River, in Costa

---

[17] Report upon the Physics and Hydraulics of the Mississippi River, by A. G. Humphreys and H. L. Abbot, Bureau of Topographic Engrs., U. S. War Dept., 1861.

Rica, A. P. Davis, Past-President, Am. Soc. C. E., arrived at percentages of 5.2, 1.7, and 7.1 for June, July, and August.[18] By comparing records at Yuma and Topock, Fortier and Blaney estimated the bed load of the Colorado at Yuma as 25% of the suspended load.[19]

Until more light is thrown on this question, a degree of uncertainty must attach to estimates based on suspended silt alone. Simultaneous vertical curves of silt content and velocity in Texas streams incline the writer to believe, however, that the quantity of silt transported as a true bed load comprises a much smaller percentage of the total load than is generally assumed.

REMOVING SILT FROM RESERVOIRS

Various plans for the removal of silt from reservoirs have been proposed, but a satisfactory and feasible method has yet to be found. One plan calls for an auxiliary basin at the head of the reservoir to catch the silt and a conduit through which it could be flushed to a point below the reservoir. Such a system, no doubt, would work with some degree of effectiveness, but its cost would be prohibitive. Another method involves stirring up the silt with a jet and allowing it to pass out with water drawn off, but this plan does not work satisfactorily even in the smallest reservoirs.

Dredging will naturally come to mind as a method with possibilities, but under present conditions the cost would be prohibitive. As favorable sites become scarcer and the cost of this method decreases through improvements in methods and machinery, it is possible that in time dredging may become economically feasible.

Some small reservoirs have been partly cleaned by draining off the stored water and permitting the flow of the stream to scour out the deposit. It is doubtful whether this method could be used with any degree of success with larger reservoirs. The stream will cut a narrow channel through the deposits, and scouring action will cease before any considerable proportion of the deposit has passed out of the reservoir. There would be a better chance for the successful use of this plan if satisfactory methods could be devised to direct the flow of water to any point desired. Another objection to sluicing is that where water for irrigation is valuable enough to warrant the construction of extensive storage works it is too scarce and valuable to be used in clearing a partly silted reservoir. At present (1929), the only feasible method of remedying the situation created by a silted reservoir is to secure additional storage capacity by increasing the height of the dam or by building a new reservoir.

PREVENTION OF SILTING

Surface soils and banks of channels under the erosive action of flowing water are the two chief sources of silt. The problem of preventing the silting of reservoirs, therefore, will involve either the elimination of, or a reduction in the quantity of, silt thus produced, or its removal from the stream above the reservoir.

---

[18] Noted in "Irrigation Practice and Engineering": Vol. II, "Conveyance of Water," by B. A. Etcheverry, M. Am. Soc. C. E., p. 87.

[19] "Silt in the Colorado River and Its Relation to Irrigation," by Samuel Fortier and Harry F. Blaney, U. S. Dept. of Agriculture, *Technical Bulletin No. 67*, 1928.

Under present conditions the outlook for effective measures to these ends is not particularly encouraging. Forest and grass cover on public lands can be maintained, but the private owner is allowed to clear his woodland, over-graze his pasture land, and permit his cultivated land to wash and add to the silt load of the streams. There is, however, a growing tendency among farmers to terrace their lands, to adopt contour plowing, and to check gullying; and as these practices become more widespread there must result eventually a substantial reduction in the quantity of silt which reaches the streams.

For reducing the quantity of silt developed by the erosion of banks, dependence must be placed chiefly on flood control, which, by holding the extreme flow of the stream down to moderate stages, will eliminate the high velocities mainly responsible for excessive erosion. Barriers of brush or other material in the smaller channels and lines of living brush across the flood-plains of larger channels, by decreasing velocities and thus encouraging deposits, should relieve these channels of a part of their load of silt. The effectiveness of a dense growth of salt cedars in desilting the Pecos at Lake McMillan is reported by Walter.[20] This growth immediately above the lake dates from about 1918 and is responsible for a reduction in the rate of silting of the reservoir from an average of 1 930 acre-ft. per year from 1894 to 1915 to an average of 350 acre-ft. per year from 1915 to 1925.

There is a possibility of desilting a stream above a reservoir by diverting the silt-laden water through canals leading out over the flood-plain and permitting the silt to deposit in natural or artificial basins before the water is returned to the channel of the stream. Such a system would be costly, but might prove feasible in particular cases.

The problem of preventing the silting of reservoirs is one which should be handled by an organization qualified to deal with a stream as a whole. Each of the desilting works or reservoirs that such an organization would construct, would be required to absorb only a part of the silt burden of the stream, and costs could be spread on an equitable basis over all those benefited. Otherwise, each reservoir would profit without cost from any desilting works or reservoirs in the stream above it.

ESTIMATING THE EFFECTIVE LIFE OF A RESERVOIR

In a study involving the effective life of a storage reservoir the first step is to determine the average silt load carried by the stream at the site. Then a consideration of the fineness of the silt, the manner in which it may be expected to be deposited, and the exposure and scouring to which the deposits will be subjected, will permit a conclusion as to the volume the yearly load of silt will occupy. The problem then becomes one of adjusting the size of the reservoir to meet the requirements for a long life at a feasible cost. In this connection, attention should be called to deposits above the flow line, the quantity of silt which may pass over the spillway, the quantity which may be removed through outlet gates, and the return seepage.

When silt-laden water strikes the back-water of the reservoir its velocity is reduced, and the coarser material is deposited immediately. This deposit continues to grow and extend up stream. When conditions are favorable, it may build up to the point where a substantial volume of deposit, which otherwise would occupy space in the reservoir, is above the flow line.

---

[20] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 1722–1723.

Samples taken on the spillway of the Austin Dam and for several months on the spillway of Lake Worth indicate that, until silting has gone so far as to produce channel conditions in the reservoir, the quantity of silt which passes over the spillway is negligible. The reservoir at Austin is so filled with silt that channel conditions prevail and the percentage of silt in the water passing over the spillway differs very little from that in the flow above the reservoir. On the other hand, at Lake Worth where silting is only moderately advanced and channel conditions do not obtain, samples taken on the spillway after the inflow of heavy floods carrying large quantities of silt rarely showed a silt content in excess of 0.02% by weight.

It is well known that in some instances when a reservoir is lowered at a relatively rapid rate water from the saturated sides and bottom seeps back into the reservoir and may amount to a substantial quantity. It is believed that deposits of the very fine material which constitutes the greater part of the silt load of Texas streams will give up water at a very slow rate, probably not fast enough to offset evaporation at its surface. These deposits should not be depended upon as an aid in tiding over critical periods.

That water drawn off through low reservoir outlets will carry some silt with it, is certain, but the quantity is doubtful. The scouring effect of the water drawn into the outlet will extend up stream only a short distance, but if the finest silt flows along the bottom of the reservoir, as was indicated at Medina Lake, Lake Worth, and Lake Kemp, there is a possibility that a substantial quantity of this fine silt will reach the area affected by the gate and be carried out.

DISCUSSION

T. U. TAYLOR,[21] M. AM. SOC. C. E. (by letter).—In describing the silting condition of the Colorado River in Texas the author mentions some observations reported by the writer in 1928.[22] Since that time an aerial photographic map has been made of the dam and the lake at Austin. It covers a stretch of the river 4 000 ft. long, including Bee Creek which is a tributary from the west. Fig. 1 is a sketch drawn with the assistance of that map. The original water surface of 1913 includes all the area bounded by the solid lines. The present channel is confined between the dotted lines when the water is level with the crest of the new dam at Elevation 51.4. The part between the solid lines and the dotted lines shows the accumulation of silt. It indicates that the silt is gradually narrowing the channel and has reduced it from a width of 1 300 ft. in 1913 to 620 ft. in 1929, with the Bee Creek cross-section ½ mile up stream. The area on the right has been reclaimed entirely and consists of about 14.6 acres of silt.

There are several striking points illustrated by this survey. Point *A* (Fig. 1) is in the original river bed where the water in 1913 was 51.4 ft. deep. This depth has all been taken and the space is now a sand beach covered with willows and weeds.

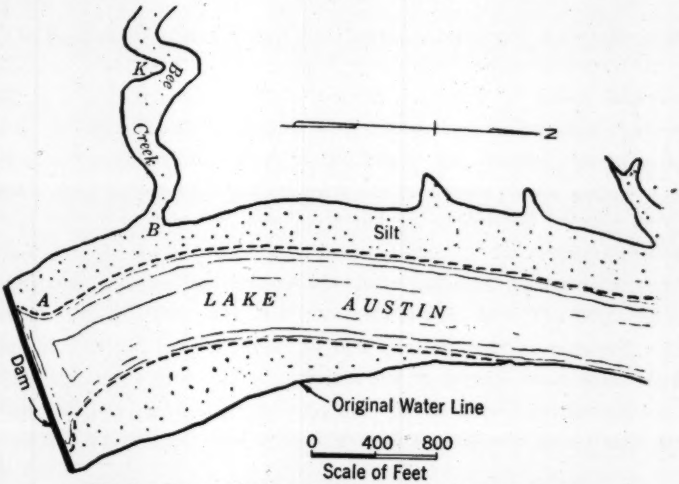


FIG. 1.—SKETCH SHOWING EXTENT OF SILT ACCUMULATION.

At Point *B*, in the mouth of Bee Creek, the water was originally 51 ft. deep in 1913. It has now a bar of silt 2.4 ft. above the lake level, making a depth of 53.4 ft. of silt.

In 1913, a boat-house was anchored at the sharp point, *K*, near the word, "Bee", and a camping party dived off the boat into 30 ft. of water. In 1930

---

[21] Dean of Eng., Univ. of Texas, Austin, Tex.

[22] "The Silting of the Lake at Austin," Univ. of Texas, *Bulletin No. 2439.*

it has become "high and dry," and Bee Creek from head to mouth is a solid plug of silt varying in depth from 53.4 ft. to zero. On the west shore, a strip of land has been reclaimed or taken from the lake by the encroachment of the silt that extends 5.2 miles up the lake, varying in width from 100 to 300 ft.

Measurements made opposite the mouth of Bee Creek in 1929 revealed a cross-section of 2 400 sq. ft. as compared with 37 300 sq. ft. in 1913. It is thought that the 1929 maximum depth opposite Bee Creek will scarcely be decreased. Other phases of this subject were discussed more fully in connection with another paper.[23]

---

[23] **"Silting of the Lake at Austin, Texas," by T. U. Taylor, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1681.**

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1785

CITY PLANNING AS RELATED TO THE SMALLER CITIES[1]

By E. A. Wood,[2] M. Am. Soc. C. E.

With Discussion by Messrs. John L. Starkie, O. H. Koch, Stephen Child, and E. A. Wood

Synopsis

The National Conference on City Planning has urged American municipalities to establish City Planning Commissions, adopt a City Plan and "sell" it to the citizenship. This program embraces all cities, irrespective of size. It can probably be applied with greater ease and greater chances of success to the smaller city than to the larger one, because there are fewer mistakes to correct and the subject-matter is in a more plastic stage.

The City Planning Commission should be established by ordinance, to furnish some degree of permanence to its existence. Its members should be selected from outstanding citizens who have sufficient leisure to attend the meetings, inspect plans, meet with interested owners, etc. The City Planning Commission should have a budget so that quarters may be acquired, a Consultant employed, and a staff of sufficient size to prepare maps, make studies, arrange meetings, give out publicity, etc. Rules should be formulated for the platting of property and a zoning ordinance should be adopted at an early date.

A Master Plan should be prepared covering not only the incorporated area of the city, but the adjacent territory as well. This should show all streets, alleys, blocks, schools, parks, parkways, streams, etc.

Popular support is always necessary for the carrying out of plans. This is particularly true in the smaller cities.

---

[1] Presented at the meeting of the City Planning Div., Dallas, Tex., April 25, 1929.

[2] City Planning Engr., Dallas, Tex.

THE PROGRAM IN GENERAL

In adopting a comprehensive program for its work the National Conference on City Planning has urged each American municipality to do three things: First, provide a City Planning Commission and an office where the Commission may take part in the City's administrative machinery; second, prepare and adopt an Official City Plan; and, third, secure public support for the administration of the Official Plan. Such a program may be adopted by any municipality irrespective of size, location, or composition. It also clearly shows that the difference in the relationship of city planning in large and small cities is one of size only; the principles are the same and legislation is equally necessary in either case. There is this difference, however, that city planning is easier for the smaller cities to undertake, is more effective, and does not cost nearly as much.

Every city should be as carefully planned as a modern railway terminal, a suspension bridge, or a central lighting plant; but the planning should be done before the city has developed. Further, it should be along regulatory lines before mistakes occur, rather than through the corrective method after errors have been permitted. Therefore, if the Official Plan is prepared in advance of city growth, its enforcement becomes a matter of public support, strengthened by the necessary rules and regulations to make it generally effective.

CITY PLANNING COMMISSIONS

As pointed out by the National Conference on City Planning, the establishment of a commission is the first step. Since private property and property rights are involved, a commission is better able to deal with the public than the Mayor and Board of Commissioners. The members of the Board do not have time to devote to the many problems that arise, nor are they politically free to treat the subject dispassionately. The Planning Commission should be created by a properly drawn ordinance to give the required official status. Preferably, the ordinance should be patterned after "A Standard City Planning Enabling Act", as prepared and published by the United States Department of Commerce.

This Act was prepared by an Advisory Committee, members of which have had many years of first-hand experience in coping with local planning problems, both as citizens and in connection with the leading National business, professional, and civic groups which they represent. During the three years' work required to draft the Act, the Advisory Committee made laborious research into legal problems and consulted with expert planners, members of planning commissions, municipal officials, and other interested persons throughout the United States. The Act, therefore, represents the very best thought on city planning, is concise yet brief, and contains the necessary legal phraseology.

PERSONNEL REQUIRED

The relationship of city planning to the smaller city should be real. It should create a lively interest on the part of every citizen. The membership of the City Planning Commission should be carefully selected and should be composed of outstanding citizens who can devote considerable time to the study of physical conditions on the ground as well as to the numerous meetings that will be required with different groups of individuals. The membership should be varied, and, if the plan is to be comprehensive and include all phases of city life, the public schools, park board, women's clubs, chamber of commerce, and retail merchants should be represented with other organizations.

Representatives from the City Government should be limited to a member of the Council and the City Engineer. The former should act as a liaison agent between the Council and the Planning Commission. The latter is entitled to membership by virtue of his office; he is also charged with the execution of the plan and to some extent with the enforcement of the Commission's rules and regulations. The Mayor and City Attorney should be members *ex officio,* the Mayor by virtue of being the head of the Council, and the City Attorney by reason of his legal training.

The ordinance creating the City Planning Commission should also set up a budget to provide for the establishment of a City Planning Office and the employment of a City Planning Consultant. The former is necessary to provide working quarters for the Commission, a meeting place, and a forum for the creation of interest in the Official Plan. Public interest in the plan is of prime importance, and it is necessary that some one explain it, exhibit maps, and discuss advantages both pro and con, if that interest is to be sustained.

Many people respond by giving counter proposals, and interest is aroused and held if the work progresses expeditiously. In the smaller city, it is easier to create this interest and hold it, because the diversity of objectives common to the large city is lacking. Also, the opportunities for correct planning are more numerous in the smaller city; at least, the physical boundaries are more elastic and are more readily moulded into the Official Plan.

However, one must ever be on the alert for mistakes. A Consultant should be employed, not only to guide and direct the planning, but also to serve as mentor to the Commission and the citizens. It is very easy to make mistakes, and early enthusiasm for city planning benefits may lead to serious errors unless held in check by the Consultant.

RULES AND REGULATIONS

As a preliminary step the City Planning Commission should adopt a number of rules and regulations. The most important of these are the platting requirements and the zoning regulations. The platting of private property is one of the most fruitful sources of errors in city planning because it is so frequently left entirely to the individual owner, who fails to co-ordinate his plans with existing streets, blocks, and alleys. This is largely due to the absence of a Master Plan for the gradual development of the city. By means of certain regulations control of the platting of new additions is placed in the hands of the City Planning Commission and gives it adequate opportunity to study and adjust the platting to fit existing conditions.

In Texas, cities of 25 000 population, or more, now have the power to control platting, not only within the city's limits, but also for a distance of five miles beyond. In smaller cities, control within the city's limits may be obtained by withholding water, sewer, or other utility connections until plats are approved by the City Planning Commissions.

ZONING AN ESSENTIAL FEATURE

Zoning regulations deal with private property and not only restrict the use of land, but also limit the height of buildings and the density of population. They meet a popular demand for protection of home neighborhoods against invasion by undesirable uses. This popularity is shown by the number and class of cities in the United States having zoning regulations, to the extent of 583, as follows;

Number of Cities with Zoning Regulations.	Population.		
53	Less	than	1 000
144	1 000	to	5 000
102	5 000	to	10 000
103	10 000	to	25 000
79	25 000	to	50 000
47	50 000	to	100 000
55	More	than	100 000

This type of regulation is legal, not only in Texas, but in a majority of the States, because it is based on the police power of the Government which seeks to protect the health, safety, morals, and general welfare of the community. This authority was given to Texas cities by the Legislature in 1927.

THE MASTER PLAN

One of the most important duties assigned to the City Planning Commission is the preparation of the Master Plan, or Guide, for the future development of the city. It becomes an easier task after the adoption of the platting rules and the zoning ordinance because these measures afford the necessary means of control. The Master Plan should show not only the incorporated area, but the contiguous area as well, and should include all streets, highways, alleys, parks, school grounds, railways, interurbans, streams, lakes, and water-fronts, if any. The area to be included should be governed by the jurisdiction of the City Planning Commission over the platting of private property. In Texas, this would be a distance of five miles beyond the city limits, but in other States this would depend on the State law.

Other features of the Master Plan would be the continuation of the building lines and set-backs as established in the city proper by means of the zoning ordinance and the street-widening program. This is particularly desirable in the suburban business districts where traffic is heavy and it is essential to keep the thoroughfares as wide as practicable.

PARKS AND PARKWAYS

Parks are an essential part of the Master Plan because they provide for recreation. The price of real estate usually requires that they be acquired well in advance of development of adjacent property, otherwise the value of the land is too high. Certain cities now require sub-dividers to dedicate 10% of the area of their sub-divisions for park purposes, a process which materially assists the development of a park system.

Parkways, boulevards, and walkways should be located along creeks or ravines. This is very important in growing cities because it furnishes storm sewers made to order. Too frequently this feature is overlooked, and natural drainage channels are filled to create additional lots or streets. As a result of the parkway adjacent property values are enhanced and natural beauty spots are preserved for future use in a boulevard system.

PUBLIC SUPPORT BUILDS THE FUTURE CITY

Popular support is necessary, and any city plan must be "sold" to the public or the Master Plan will never be carried out. Experience in many cities bears out this statement. It is sometimes said that it is better to have "more selling and less planning". The essential point is to convince the community as to the advantages of planning for the future. Citizens will then demand a plan before they will vote bonds for unco-ordinated work. This is particularly true of the smaller city, where public funds are limited.

DISCUSSION

JOHN L. STARKIE,[3] M. AM. SOC. C. E.—It seems to be commonly accepted that every city, irrespective of size, is subject to the same general principles in the application of a modern, comprehensive city plan. There are few, if any, problems present in the larger cities that are not also encountered—although most always in lesser degree—in the smaller communities. An economic advantage, however, rests distinctly with the smaller city, in that the earlier in the state of development of a city the plan becomes operative, the greater will be the saving in cost and the simpler the task of the city planner. In meeting an incurable obstacle in the preparation of a city plan or one surmountable only at prohibitive cost, what engineer has not dreamed of the ideal condition in which a city has waxed and expanded from the date of its inception in accordance with a well prepared and well directed city plan? However, such attempts as have been made in that direction may well be termed "noble experiments".

It is fortunate, perhaps, for the progress of the science of city planning in America that the advancement of a city under planless processes must usually reach the point where the resulting chaotic and wasteful conditions awake public consciousness to a realization of the need and economy of an ordered and directed procedure in conformity with a carefully thought out and scientifically prepared plan. This is good because it presents a variety of difficulties to overcome which make for progress in the perfection of the science, and it enlists initially the much-to-be-desired interest of the citizen.

The problem of securing and holding the interest of a larger percentage of the citizens is usually less of a task in smaller cities. However, there is more of a tendency to urge the engineer to incorporate in the plan ideas that are supposedly advanced for the alleviation of existing faults, but with no outlook on the future.

Many merchants in the smaller cities regard a traffic congestion on the streets in the vicinity of their places of business as a sure sign of progress and a condition to be preferred. Fortunately, however, highway engineers in Texas are insisting on locations from a broader viewpoint, so as to by-pass through traffic around population centers and, at the same time, to avoid railroad grade crossings wherever possible.

The traffic problem is present everywhere, in the small as well as in the large city, and the elimination of railroad interference with thoroughfares occupies a very important part in considering that question. From a long experience in railway service the speaker has no hesitancy in stating that practically every reasonable request, equitably made, to share in the expense of grade-crossing separations on existing locations of railways will meet with ready and co-operative response on the part of the railroad so far as financial abilities will permit. There is an alarming increase in the number of new grade crossings being opened, as compared with the number that are being eliminated. The city planning engineer who specifies, with every railroad grade-crossing separation which he proposes, the closing of one or more existing grade crossings, serves humanity in general as well as the city which employs him.

---

[3] Office Engr., G. C. & S. F. Ry., Galveston, Tex.

A statement by the Master Plan Engineer of the City of Dallas that "planning should be done along regulatory lines rather than the correction of existing errors", indicates an active interest in the whole success of city planning. The 1927 Enabling Act of Texas also takes cognizance of this viewpoint. Any city plan which proposes in whole or in part for immediate consummation, changes, alterations, or relocations, at a wholesale rate, is untimely. A city plan once adopted should remain in operation perpetually (subject to such modifications as changing conditions demand), and the accomplishment of its objectives should be made a gradual process with its complete realization possibly never fully attained. Thrifty railroad companies are continually changing their standards of various physical properties, but, except in the rarest of instances, the new facilities are not installed until the old ones have completed their useful service life, or until unusual economic conditions warrant a change. Adaptation of this thought to city planning is illustrated in the "set-back building line". The process may be slower, but the transition to the accomplishment of the plan is performed in an orderly manner; it interferes least with the even tenor of the industrial, commercial, social, and moral life of the community, and, in the end, it provides a more enduring continuous and useful service to the greatest number of citizens.

O. H. KOCH,[4] M. AM. SOC. C. E.—That city planning is just as vital and important to the smaller cities as to the larger ones has been emphasized by the author. He has outlined a number of principles and recommendations as regards city planning in general. The speaker would like to call attention to a few points wherein planning in the smaller cities differs from that in the larger cities.

A population of less than 50 000 may be considered to define a small city. This is the type that predominates in Texas as well as in other States. Texas has many cities of less than 50 000 inhabitants, but only a few that are larger. The same type of discussion could be very readily carried on as to the difference in planning between cities of 100 000 or 200 000 inhabitants and the great metropolitan centers. In each of these various classes there are many local and individual problems which are peculiar to each classification, and which need to be solved in a different manner. Many people have the impression that city planning, in a town of 25 000 to 50 000 inhabitants, is a comparatively simple matter, but the speaker maintains that a good, sound, sensible, and practical city plan for a town of this size requires a great deal of study and careful consideration.

The first step in preparing a city plan is to take an inventory of the existing conditions. This step in a small city is, of course, much easier and more simple than in the larger cities. The next step is to analyze the projected needs of the community by studying its past record of growth and to attempt to predict the future growth and needs. In this prediction it will be necessary to determine the probable type and character of the future of the community. This problem of trying conscientiously to predict the future trend of growth has caused the speaker more concern in plans for the small city than it has for the large one, and that particular decision is one of the most important essentials in preparing any city plan. City planning should be at least 90% preventive and 10% corrective. The accuracy with which the prediction of future growth is made will determine, to a great extent, the ultimate value of that city plan.

---

[4] Cons. Engr. (Koch & Fowler), Dallas, Tex.

It is relatively easier to make such predictions in the large cities because of the availability of a more definite record and tendencies of growth upon which to base predictions. In the small cities this particular information is somewhat limited, and the planner has, as it were, a shorter base line from which to project and predict future tendencies. In the large, more established cities, the trend of growth and type of development is more firmly established and its general character is more permanently fixed. Any unusual development would not change its established characteristics to any great extent compared with the material effect that the same unusual development might have upon the smaller city where the proportion of importance would be much greater. In this type of city it is possible, by the introduction of a large project, to change the entire character of its development.

For example, in a small city, supported primarily by educational institutions, and which has been residential in nature for many years, a satisfactory city plan could be developed based upon such conditions. Suppose that a large manufacturing company should decide to build its factory in this city and to import a great number of factory employees. This unusual development in such small city would tend to unbalance the entire situation; it would change its character and would require a complete change of the plan. In the larger city the introduction of a fair-sized manufacturing plant would not make any appreciable change, since it would probably not be the first manufacturing plant in that city. That phase would have been anticipated to a certain extent, and the city plan could be adapted to meet the situation without materially affecting its character.

Thus, in a smaller city, it is necessary to decide whether or not it would be advisable to prepare a plan providing for such possible industrial developments, thereby furnishing incentives and encouragement for securing additional activities for the city, or whether it would be better to ignore such possibilities and prepare a plan which is limited to developments of its present type, thereby discouraging the possibility of securing such industrial activities.

Finance is always a subject of paramount importance to be considered in the city plan. While the first cost of engineering studies for a plan in the small city is not as great as in a larger city, the per capita cost would probably be more. This expense, however, is not a minor item considering the usual restricted budget of the small city. Even if the per capita cost were the same it would be much more of a burden in the smaller city. The same thing is true in the matter of corrective city planning. While the property values are not as high and the corrective measures would not total the large amounts which are reached in the greater cities, it is also true that smaller cities have less ability to pay even a smaller proportional cost. This condition is a great stumbling block and a real situation which must be faced. It requires considerable judgment in order to inaugurate and execute what would be thought to be a practical and economical program, both from the standpoint of first cost as well as that of executive cost.

The chief value of city planning lies in anticipating future needs, which is a matter of budgeting the city's facilities, much in the same manner as its financial program is budgeted. If the planner is skeptical in regard to the city's growth and tries to be too conservative in planning for the immediate future only, in order to hold down cost, he will not be securing the full measure of benefit to be derived from city planning. On the other hand, if he is too optimistic and plans for too many years in advance he is apt to cause exceedingly large expenditures for immediate use and to have excess of expense and unused facilities. The excess cost of the superfluous facilities, the overhead carrying charges, and the maintenance of these items, capitalized over a number of years, could very readily exceed what the cost would be with a more conservative plan, even if the additional corrective measures had to be taken after a period of years. It is this choosing of the happy combinations that will determine the comparative value which any city plan may develop.

In the larger city the cost of future corrections, after a long period of years, would be considerably more in proportion than the probable cost of such additional widening would be in a small city. This fact, together with the greater valuations and borrowing power on the part of a larger city, would justify more liberal plans and ambitious programs than it would in the small city. For example, the cost of excessive widths of the streets in a small city might be a burden, even if such cost were limited to present expenditures for excess right-of-way widening only. Even though the paving on such streets was not unusually wide for the present, the cost of paving intersections, extra length of driveway turn-outs and cross-walks for each residence, as well as the additional maintenance for such extra paving and walks, would have to be considered. While these items may seem comparatively insignificant to planners in the larger metropolitan areas, they are a much greater proportion of the original project than in the larger city, and this excess cost, capitalized and put on compound interest over a period of 25 or 30 years, would pay for a great deal of corrective work at that time.

In the larger cities playgrounds, parks, etc., can be provided with much more flexibility because of the greater number to be supplied; whereas in the small cities the matter of first cost per capita again enters into the economic amount which can be spent for such facilities. In the small city the density of population per acre is considerably less than in the larger city. The facilities required for playgrounds and parks would be less and, if sufficient playground space were provided, based upon a liberal estimate of the density of population, several years in the future, it would add an unnecessary immediate burden upon the city in first cost as well as in maintenance charges.

In the matter of zoning, the areas set aside for business, industrial, and residential purposes would vary considerably in their proportion to one another depending upon whether the problem related to a small or to a large city. In the small one there is not so much possibility of providing transitional property that could be readily changed to permit of the expansion of the business and industrial sections.

In the small city the matter of railroad grade elimination must be handled in an entirely different manner. The small communities are not justified in spending money to eliminate the objectionable grade crossings. This problem, which is often planned in the larger cities as a major operation, must be reckoned with in an entirely different manner in the small city. These same principles are typical of the other phases of city planning.

While city planning is of great value to the smaller cities, the real value will only be obtained by a practical and common sense application of its principles heretofore developed, scaled down to the proper proportions to suit the idiosyncrasies which are peculiar to the small city. City planning in the beginning was born of necessity due to congestion and intolerable conditions in the larger cities. Most of the pioneer work in this field was developed in order to relieve such conditions. These obvious advantages and the favorable results have been made possible through city planning; and these acute and intolerable conditions first gave city planning its importance. The relief gained has encouraged the city planning principles to be applied to the preventive phase, and has educated the public to the realization of the great advantage of planning in advance. This work has been most intensive in the larger cities and the methods and principles adopted were designed to suit conditions there. With the advantages thus favorably established the move naturally is spreading into other cities as well. The success and degree of benefit which the smaller cities will receive, will depend entirely upon the city planner's ability to recognize the slightly different conditions; they will depend on his sense of proportion in adapting the principles, in a practical manner, to suit the differing conditions.

STEPHEN CHILD,[5] M. AM. SOC. C. E. (by letter).—Mr. Wood reiterates the recommendations of the National Conference on City Planning that all American municipalities should establish city planning commissions. He states, furthermore, that provision should be made for the establishment of a City Planning Office and the employment of a City Planning Consultant. Unfortunately, smaller cities frequently become imbued with the idea that it is quite unnecessary to have a consultant, perhaps because a local architect and engineer are members of the commission. This is false economy.

City planning is a never-ending job. Cities cease to need planning only when they cease to grow. There are many methods of procedure and no "hard-and-fast" rules established. A great metropolitan community must adopt the method best adapted to its own peculiar conditions. However, a large group of cities, ranging in population from 10 000 to 100 000, have much in common as regards the more general features of city planning procedure.

---

[5] Landscape Archt.; Consultant in City Planning, San Francisco, Calif.

For example, they first need a properly organized city planning commission; then a comprehensive civic survey, to be followed by the preparation of plans for streets, zoning, parks, and other features of growth. Furthermore, and perhaps most important, all these plans and projects must be followed up patiently and continuously, year after year, until finished.

For most of the smaller communities, a separate city planning department would be out of the question. The necessary "follow-up" work is no doubt done best "at home" and not from some expert's office. Recognizing this essential, more or less complete transfer of city planning functions from the expert's offices to some one of the city's departments, it may be best to begin in another way and to proceed with the plan by fundamentally different methods.

As an example of how many mistakes are made in city planning, consider a hypothetical small town. Suppose that, under the stimulus of a campaign carried on by an effective citizens' advisory committee and sponsored perhaps by the Chamber of Commerce, a city planning commission has been organized, a fund appropriated for its work, and a competent consultant employed. For a year or more everything seems to be progressing quite well. A preliminary survey is prepared; an illustrated pamphlet report, containing recommendations for a major street plan and for other important features controlling the city's growth, is published.

Then comes the inevitable slump. Opponents of the zoning plan appear; the Committee declines to appropriate money for a much needed traffic artery; a proposed bond issue to provide for a playground system, park, or civic center is voted down; and, finally, the funds for the city planning commission itself are either not appropriated at all or are radically cut.

By this time the expert has gone on to new fields of endeavor. Possibly some of the valuable data and plans he has prepared are buried in the vaults of the city clerk's office and forgotten. Much material remains in the possession of the expert, whose office is perhaps a thousand miles away. The city then drifts along in practically the same shiftless, inefficient manner that it has for years. Small and large subdivisions continue to be added and haphazard growth goes on, with little guiding or planning.

Then follows another "spasm" of civic righteousness—the result, perhaps, of some grade-crossing accident or other occurrence that arouses public interest. There is a scurrying around in search of mislaid data and the original expert and city engineer are not now near at hand to help in the re-educating work.

The foregoing remarks will serve as an example of conditions that are working against good city planning, particularly in the smaller cities. The ideal steps proposed for the same situation might be analyzed somewhat as follows:

Consider a city of about 30 000 population. Often with the support of the Chamber of Commerce, a Citizens' Advisory Committee is appointed, and this Committee employs a consultant who helps in the preparation and enactment of city-planning ordinances. The ordinances will include the general features proposed by the Advisory Committee on City Planning and Zoning of the United States Department of Commerce as recommended by Mr. Wood. This step in itself would take a little time and, in the meantime, the Consultant will give some talks on city planning. Ultimately, a city planning commission is authorized and organized.

Under the terms of the ordinance the City Engineer is made an *ex officio* member of the City Planning Commission. Contrary to the example previously cited, the City Planning Commission does not turn the job over to the consultant. On the contrary, it selects an efficient secretary—perhaps a stenographer in the office of the City Engineer. She is to begin immediately to have charge of the data collected. The City Engineer will probably need to employ a city planning assistant. Larger communities might need more than one, and there is the possibility that one of these might become secretary to the Commission. The Commission, again under the advice of its Consultant, proceeds to make a comprehensive civic survey in such a way that every member will be familiar with it. They will be able to amend, extend, and perfect it and they will have all its details available for use in future years.

The next step undertaken by the members of the Commission is to prepare a comprehensive city-planning program. They will endeavor continuously to keep the program of planning up to date. As a whole, the program will need to be flexible enough to permit adjustments to new conditions. One important requisite is the contract drawn between the Commission and the City Planning Expert. This should include such matters as fees, the time to be spent by the Consultant, the extent to which he shall co-operate with the City Engineer, and his duties in respect to directing the civic survey and any field investigations which may be necessary.

Suppose the Council should reject a street-widening proposition, thus upsetting the major street planning; or, suppose the people should vote against a bond issue for parks or a civic center. Every one knows that these are only temporary setbacks. Sooner or later, if the Commission's plans have followed comprehensive civic surveys and are really well considered, these projects will have to be carried out, perhaps in some modified form. If all the data are at hand and the Consultant under the terms of his contract is also at hand, he can insure that these modifications are reasonable and in accord with the general program prepared by the Commission. It is believed that this second proposed method of procedure will go far toward solving the problem of overcoming city-planning inertia which is often a serious matter.

E. A. Wood, M. Am. Soc. C. E. (by letter).—The discussion which has followed the publication of this paper has brought out a number of interesting suggestions. Most of these deserve careful consideration and final adoption by the City Planning Profession. Probably the suggestion having the greatest merit is the one made by Mr. Starkie concerning the separation of grade crossings.

Grade-Crossing Separation.—Mr. Starkie suggests that, for every grade crossing shown upon the Master Plan for ultimate separation, additional grade crossings should be shown for closing or complete abandonment. This seems to be a very efficient and practical manner of dealing with this subject. It also holds out the hope that complete solution is in sight, rather than the inevitable multiplication of future separations or the removal of entire railway lines, as so often suggested in city plans.

Railways have their place in communities in much the same way as highways. Part of their difficulties have come about through improper platting of subdivisions and a demand for grade crossings which have resulted from the frequency of dead-ended streets. City planning can easily take care of this feature in the future, but the work that has already been done must be dealt with in the fashion suggested by Mr. Starkie. Assuming that the railroad is a barrier, it is better to make it a safe barrier and prohibit all crossings except those called for in the Master Plan, than to make it a dangerous barrier and subject to daily loss of life.

Inventory of the City.—Mr. Koch stresses the necessity of taking a complete inventory of the city before any planning is done. This is exceedingly important and furnishes the groundwork for city planning. It also furnishes the city planner with the incentive to make each city somewhat different from a preceding city. It is a mistake to attempt to mould American cities after a standard pattern. There is a wealth of history, tradition, and environment to draw upon if the city planner will only take the time for investigation. Each factor should leave a particular stamp upon the Master Plan.

This particular phase of city planning also has a distinct appeal to the average citizen. If properly used, it will furnish a means of contact with the citizenship that will be invaluable to the planner. It will stimulate interest in the plan, increase attendance at meetings, and furnish human interest, subject to development by the press. It will popularize city planning in the smaller cities.

Regulatory Planning versus Corrective Planning.—Mr. Koch also stresses the cost of city planning and the inability of the smaller city to pay for parks and street widening. This same cost proves the wisdom of acquiring parks by gift and of widening streets by the set-back line rather than by purchase of property. The control of the platting of property furnishes the means of acquiring, not only park property, but school property as well. It is one of the most valuable means for good city planning. It serves the community, and it serves as a common meeting ground for School Boards, Park Boards, and City Plan Commissions.

By a judicious use of the set-back, practically the entire Master Plan can be protected against encroachment and all streets made ready for widening. This does not have reference to the set-backs established in the zoning ordinance, which are done for the purpose of protecting the health, safety, morals, and general welfare of the community. It means those set-backs which are laid down by ordinance for the express purpose of preventing the erection of buildings on streets that are to be widened at some future time. This type of regulatory planning is well adapted to the smaller city because it does not involve the expenditure of public funds. One of its chief benefits is that it invites co-operation between citizens and the municipality.

Continuity of City Planning.—Mr. Child suggests the possibility of the lack of interest in city planning after the completion of the Master Plan and the danger of its utter failure. This possibility is always present in the smaller city, and every means should be taken to guard against it. One of the reasons for making the City Attorney and the City Engineer *ex officio* members of the City Planning Commission is to develop their interest to the point where they will follow city-planning ideals in their municipal work. For the same reason it is well also to have the Mayor as an *ex officio* member of the Commission. As a progressive citizen, he not only can sell city planning to the citizenship, but he can put it into effect as well.

One of the best means for insuring continuity in city planning is to make the various plans effective through ordinances rather than by the expenditure of public funds. For instance, an ordinance requiring the submission of all plats of property to the City Planning Commission before they may be filed for record will at least furnish the opportunity for suggesting gifts of park and school lands. Such gifts may have a string tied to them, such as immediate improvement. This will be within the means of the municipality, whereas the purchase of the property would not. In the same manner the prevention of the erection of new buildings beyond the set-back line on a major thoroughfare is within the means of the municipality, while the immediate widening of the thoroughfare is not. Eventually, the street can be widened because there will be very few buildings to be torn down or moved back. There is even the possibility of the property being given to the city for the widening when the necessity becomes urgent.

City Planner Must Be a Leader and a Teacher.—If the city plans for the smaller cities are to be fufilled the Planner must be a leader as well as a teacher. He will find that the average citizen will respond to leadership. He will also find that, when the principles of city planning are thoroughly inculcated in the citizenship, plans will not die, either through neglect or by suffocation through an accumulation of dust. Much will depend upon the City Planner. He must leave his imprint, not only upon the city plan, but upon the citizenship as well. The opportunity for doing this is much greater in the smaller city than in the larger one.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1786

IMPACT IN HIGHWAY BRIDGES

FINAL REPORT OF THE SPECIAL COMMITTEE[1]

WITH DISCUSSION BY MESSRS. SEARCY B. SLACK, LLEWELLYN N. EDWARDS, AND THE IMPACT IN HIGHWAY BRIDGES COMMITTEE

TO THE AMERICAN SOCIETY OF CIVIL ENGINEERS:

The Committee was appointed in 1922, and was asked to search for available data on the subject of impact in highway bridges, to encourage experimental work in a more systematic manner than had yet been attempted, and to report to the Society concerning the available information and its application to the design of highway bridges.

Subsequently, progress reports have been presented at each Annual Meeting and published in *Proceedings*.[2] Two of these reports[3] contain pertinent data of value.

This report undertakes to present such conclusions as the Committee has been able to make from the available data; also, in the form of appendices, abstracts of such data as will assist those who are especially interested to make independent applications of the information and to direct future work along the most profitable lines.

The only data published prior to 1922, so far as the Committee could ascertain, consisted of a number of theoretical discussions and the results of experimental work by F. O. Dufour,[4] M. Am. Soc. C. E., made in Illinois from 1910 to 1912. By reason of the revolutionary changes in highway transportation and in the design of highway bridges that have taken place since these experiments were made, the data obtained at that time are of but little value in the present study, and have been utilized only in the construction of Fig. 1. The only other useful information which the Committee has been able to find is from two sources:

---

[1] Presented at the Annual Meeting, January 16, 1929.

[2] *Proceedings*, Am. Soc. C. E., March, 1923, Papers and Discussions, p. 457; March 1924, Society Affairs, p. 260; March, 1925, Society Affairs, p. 126; March, 1926, Papers and Discussions, p. 442; March, 1927, Society Affairs, p. 120; and March, 1928, Society Affairs, p. 181.

[3] *Loc. cit.*, March, 1923, Papers and Discussions, p. 457, and March, 1926, Papers and Discussions, p. 442.

[4] *Loc. cit.*, October, 1926, Papers and Discussions, pp. 1737–1742; *Journal*, Western Soc. of Engrs., Vol. 18, 1913.

(1) Studies on highway bridge impact at the Iowa State College, Ames, Iowa. These studies were begun in 1922 as a co-operative project between the United States Bureau of Public Roads, the Iowa Highway Commission, and the Engineering Experiment Station of Iowa State College. In 1924, very considerable assistance was rendered directly by the Society, through this Committee, by providing for its use a McCollum-Peters electric telemeter,[5] or strain-gauge, for determining and recording various stress measurements. Reports from this project have been published.[6]

(2) Experiments of the U. S. Bureau of Public Roads on impact on pavements. These results have also been published.[7]

The report is primarily an effort to analyze and correlate the data obtained from these projects and to draw such conclusions as seem to be warranted at this time.

Nature of the Problem

Obviously, the most direct method of determining dynamic stress and, therefore, impact in a structure is by direct measurement of such stress under moving loads subject to the various conditions met in practice. To a certain extent this is practicable, and much information of this nature was obtained in the tests at Ames, Iowa. This method, however, has its limitations. Not only are the variations of loading, truck design, and tire conditions very great and not easily defined and related, but the structure itself introduces many complications. Smoothness of floor, nature of possible obstructions to be anticipated, the design of the structure itself, and the length of roadway to be loaded for maximum effect, all enter into the problem and seriously complicate it. To cover all these variables by tests in the field is practically impossible. In particular is it difficult to determine the significant impact for those members of a structure that require a considerable load. As a consequence, the tests at Ames were directed primarily to floor-beams and stringers. Impact values were determined in a few truss members of a number of bridges, but the results are of less importance, as in most cases the simultaneous unit stresses were very low.

The greatest impact of a wheel passing over an obstacle may be from shock as the wheel strikes the obstacle, or from drop as it again strikes the floor. According to the available data, drop impact is generally the greatest for heavily loaded trucks. Shock impact was frequently greatest for unloaded trucks, but the resultant stresses were so low as to suggest that shock impact is of theoretical, rather than practical, interest.

As compared to railway bridge impact, the problem is a very different one, especially as regards truss members. In railway bridges, the impact effect arises largely from the rotation of unbalanced drivers; and synchronism of driver rotation and bridge vibration produces dynamic stresses which are subject to certain fairly well-defined laws. Such synchronism of impact can hardly occur in highway bridges when heavily loaded with several trucks. Furthermore, the observations necessary to determine the impact due to accidental obstructions when the bridge is loaded in this fashion are almost impossible.

---

[5] *Bulletin No. 247,* U. S. Bureau of Standards.

[6] *Bulletins 63* and *75,* Eng. Experiment Station, Iowa State Coll.; ***Public Roads,*** September, 1924.

[7] *Public Roads,* March and December, 1921, June, 1926, and August, 1928.

In addition to the direct results by tests on bridges, valuable information can be deduced from impact tests on pavements such as those conducted by the U. S. Bureau of Public Roads. To enable such results to be utilized, it is necessary to determine, first, the relation between the wheel blow on a pavement, and the blow on a bridge floor, under the same surface conditions; and, second, the actual dynamic stresses produced in the bridge member by a given wheel blow. The relative force of a blow, in percentage of static load, delivered by a truck wheel passing over an obstruction depends on the speed, the height of the obstruction, the flexibility of the tires, the percentage of the unsprung to the total weight, and the flexibility of the pavement upon which the wheel drops. It is less on a concrete bridge floor than on a concrete pavement; and limited data indicate that it may be less on certain types of non-rigid pavement than on concrete. The higher the percentage of unsprung weight, the greater will be the ratio of the blow to the static load; and, within certain limits, the greater the speed, the greater the blow. Furthermore, due to inertia and the time element of deflection and deformation, the stresses produced in the bridge member by the force due to a falling wheel is considerably less than would result from a static load of the same amount as this force. All these factors have been taken into account in the analysis of the data and the formulation of the conclusions of this report.

DEFINITIONS

By reason of the numerous quantities and ratios involved in the problem, it is desirable to define carefully some of the terms used, as follows:

Dynamic Force.—The dynamic force of a wheel blow is the maximum pressure of a truck wheel upon the pavement or bridge floor when the truck is in motion.

Impact Increment of Dynamic Force.—This increment is the amount by which the dynamic force exceeds the weight of the wheel. It is commonly expressed as a percentage of the static weight.

Impact Increment of Stress.—This is the amount by which the actual stress due to the moving load exceeds the static stress. It is also commonly expressed in terms of percentage of static stress.

Stress Ratio.—Stress ratio is the ratio of the actual dynamic stress produced in a member to the stress that would have occurred if a static load equal in magnitude to the dynamic force were applied at the same place.

Sprung Weight of a Truck.—This weight includes all weight carried by the springs.

Unsprung Weight.—This consists of the remaining weight: Wheels, axles, housings, and springs.

EXPLANATION OF THE DATA

On the Ames project, work was done on five steel bridges with concrete floors (four with steel stringers and one with concrete slab between floor-beams) and on seven steel bridges with timber floors on steel stringers. The individual loads with few exceptions were 3½-ton Liberty trucks, which weighed about 5 tons and were loaded with various amounts up to 10 tons, making a maximum total load of 15 tons. The tires were solid dual rubber, worn down so that their deflection was 0.19 in. under a static load of 10 000 lb. Speeds were attained up to 15 miles per hour. Stress measurements were taken on the stringers, floor-beams, and a few truss members by various extensometers; and the force of the blow of the wheel on the floor was determined (part of the time) by accelerometers.

The U. S. Bureau of Public Roads carried on extensive tests in co-operation with the Society of Automotive Engineers and the Rubber Association of America on impact on pavements from trucks equipped with various kinds of tires. The force of the blows was measured by accelerometers, and the load was passed over obstructions of various heights and at various rates of speed. Impacts were determined on smooth concrete pavement and also on old stone block pavement. Tests were also made by the Bureau of Public Roads on the relative impact on pavements of different kinds, including concrete, brick, and various bituminous pavements.[8]

With the assistance of Professor E. B. Smith,[9] an interpretation has been made of the 1921 "copper cylinder" impact data secured, under his direction, by the U. S. Bureau of Public Roads, along lines suggested in his papers[10] descriptive of those results. This interpretation has been made more readily and more clearly by comparison with similar data taken from *Bulletin 75* of the Engineering Experiment Station of Iowa State College.

In order to adapt to the determination of impact on bridges, the mass of data of the Bureau of Public Roads giving impact on pavements, the Engineering Experiment Station of Iowa State College made a few tests during the summer of 1927, in which the force of the blow from the same loads and speeds were determined upon pavements and upon a number of bridges. These latter data have not been previously published, but are included in Appendix D with the consent of the Station.

Appendix A contains a further description of the Ames tests, with abstracts from *Bulletin 75* of the Iowa Engineering Experiment Station. In this appendix are selected diagrams showing the actual results obtained on impact stresses at various speeds and under various conditions; also the impact stress as related to the dynamic force exerted on the bridge floor. The relation between the stress produced by dynamic force and static force of the same amount—the "stress ratio"—is also shown by diagrams. The selected data for impact stresses in trusses show very clearly the relatively small impact where the structure is sufficiently loaded to produce stresses of as much as 5 000 or 6 000 lb.

---

[8] *Public Roads,* June, 1926, and August, 1928.

[9] Of the Division of Tests, U. S. Bureau of Public Roads, until the summer of 1925 and since that time Research Professor of Mechanical Engineering, Iowa State College, Ames, Iowa.

[10] *Public Roads,* March and December, 1921.

per sq. in., and illustrate the difficulty involved in this part of the impact investigation.

Appendix B contains extracts from the published reports of the tests of the U. S. Bureau of Public Roads on pavements. These illustrate very well the effect of variations of conditions.

Appendix C contains results of an analysis by the Committee to render possible the approximate determination of impact stresses in floor members in terms of the separate variables involved, namely, the percentage of unsprung weight of the truck, the height of the obstruction, the speed of the truck, the relative tire deformation, and the stress ratio. It is thought that such an analysis, while having no pretention to great accuracy, will assist engineers in making an independent estimate of impact, particularly those who may undertake future experiments.

Appendix D is an analysis of the relation between impact on a concrete pavement and impact on a concrete bridge floor, and helps to relate the work of the Bureau of Public Roads previously mentioned to the Ames impact studies. It is interesting to note that the ratio of impact on a concrete bridge floor to that on a pavement when the bridge members are stressed to 8 000 lb. per sq. in., or more, does not exceed 80 per cent.

IMPACT IN FLOORS OF HIGHWAY BRIDGES

Concrete Floors.—A brief consideration of the variables involved shows how extremely difficult it is to reach any definite conclusions as to what impact factor should be used in the design of highway bridge floors.

The results obtained by application of the analysis given in the Appendices are illustrated in Table 1. The term, "worn solid tires," represents the hardest worn solid tire that has been noted. For it the value of d (the deflection of the tire due to a static load of 10 000 lb.) = 0.1 in.; for "new solid tires", $d = 0.6$ in.; and for "pneumatic tires", $d = 2.0$ in. The "normal load" consists of a truck with live load equal to the rated capacity, for which p (the percentage of unsprung weight to total weight) = 25. "Over load" represents an overloaded truck—for example, one weighing 5 tons carrying a live load of 10 tons, making a total of 15 tons and proportioned so that $p = 15$.

TABLE 1.—PERCENTAGE OF CALCULATED IMPACT INCREMENTS OF STRESS IN HIGHWAY BRIDGE FLOORS FOR TRUCKS.
(Speed, 15 Miles per Hour.)

Obstruction.	WORN SOLID TIRES.		NEW SOLID TIRES.		PNEUMATIC TIRES.	
	Normal load.	Over load.	Normal load.	Over load.	Normal load.	Over load.
None	22	12	8	4	4	2
1 in. by 2 in.	210	102	57	30	30	12
2 in. by 4 in.	475	275	122	58	50	30

The values in Table 1 cover a wide range of conditions, and show what high impacts may be expected in some cases. The Committee does not propose this table as a suggested standard, but presents it as a reasonably accurate picture of the effect of various factors on the impact on highway bridge floors. It has been computed from Equation (1), Appendix C, and the stress ratio curve of Fig. 8, Appendix A.

Timber Floors.—The limited work on timber floors with steel stringers indicates that the best of timber floors will compare favorably with clean concrete floors in regard to impact. It also indicates that on the roughest timber floors investigated the impact was about the same as for 1-in. obstructions on concrete floors.

Recommendations for Impact in Floors of Highway Bridges.—As a basis for its recommendations, the Committee has considered the following facts:

First.—That stresses due to static loads and to impact are important, as regards the safety of the structure, only when they approach design values.

Second.—That the percentage of impact increment decreases as the loads increase, and, therefore, as the unit stresses increase.

Third.—That the larger impacts observed in the tests were produced by obstructions, such as would be accidental and infrequent under actual traffic conditions.

Fourth.—That the actual occurrence, on a bridge, of loads having a magnitude corresponding to those used in the design of modern structures is infrequent.

Fifth.—That the simultaneous occurrence on a bridge floor of a maximum truck load and an accidental obstruction capable of producing high impact will be such a rare coincidence that presumably the factor of safety will usually provide safety for this condition.

This Committee recommends, therefore, that for the design of highway bridge floors and floor-beam suspenders, the impact increment of stress be assumed as 25% of the live load stress. It should be used only when the floors are sufficiently smooth to conform to good modern practice; and unusual conditions should be provided for in accordance with the judgment of the individual designer. The Committee believes that this report contains information, with necessary precision, for guidance in unusual conditions.

IMPACT IN TRUSSES OF HIGHWAY BRIDGES

The published results for impact in the trusses of highway bridges are of interest and value more for the comparison between computed and observed static stresses than for impact. The many experimental results for impact are doubtless correct within a reasonable precision, but they are not representative of the conditions that control in the design of bridges.

The results have been studied in various ways and are reproduced in Fig. 1 for trusses with concrete floors and in Fig. 2 for trusses with timber floors. Evidently, the impact decreases as the unit stresses increase. It is apparent from a practical standpoint that impact is important in design only as the combined stresses due to dead loads, static live loads, and impact approach design stresses. Such stresses have not been reached or closely approached in known experiments.

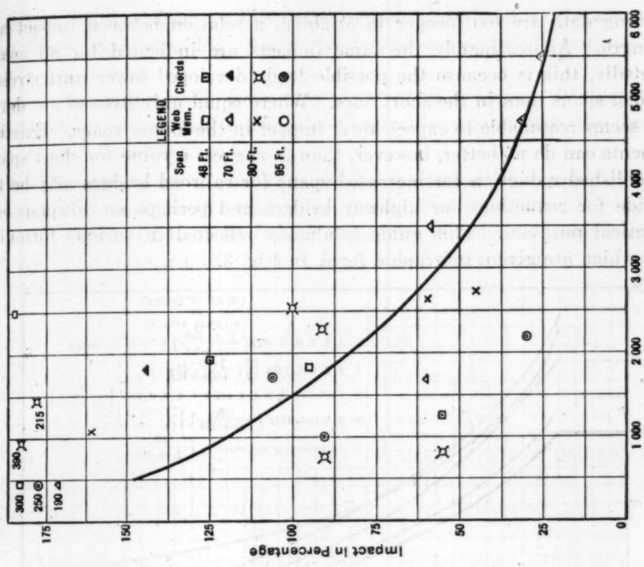


FIG. 2.—IMPACT IN TRUSSES OF HIGHWAY BRIDGES WITH TIMBER FLOORS.

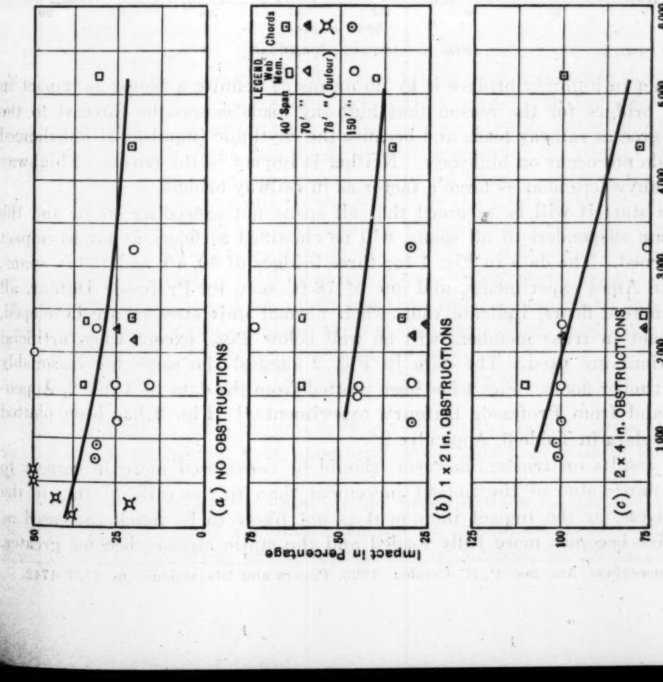


FIG. 1.—IMPACT IN TRUSSES OF HIGHWAY BRIDGES WITH CONCRETE FLOORS.

Existing data are too meager to establish a relation between impact and span length. Approximately the same impacts are indicated for all spans. Undoubtedly, this is because the possible loads developed lower unit stresses in the long spans than in the short ones. Where equal unit stresses are developed, it seems reasonable to expect lower impact in the longer spans. Existing experiments can do no better, however, than to suggest a value for short spans. The established reduction for increased spans for railroad bridges may be the best guide for reductions for highway bridges and perhaps an adequate one for practical purposes. This guide is already reflected in various formulas, some of which are given, in graphic form, in Fig. 3.

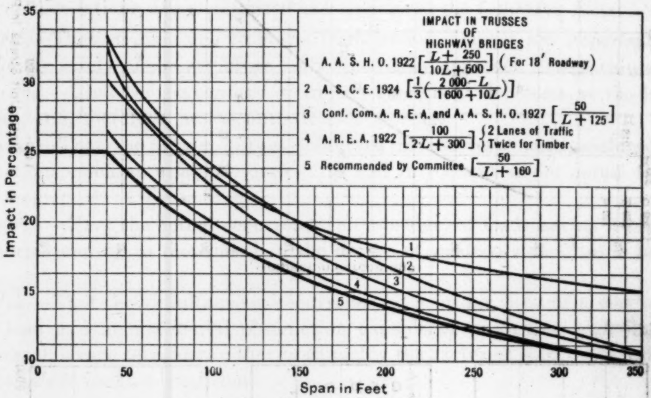


FIG. 3.—IMPACT FORMULAS.

Impact in highway bridges is by no means as definite a factor as impact in railway bridges for the reason that highway loads cannot be forecast to the same degree as railway loads and because the rhythmic impulses of unbalanced drivers do not occur on highways. Neither is impact in the trusses of highway bridges anywhere near as large a factor as in railway bridges.

As a start it will be assumed that all spans not exceeding 40 ft. and the floor-beam suspenders of all spans, will be classified as floors as far as impact is concerned. The data in Fig. 1 for three bridges of 40, 70, and 150-ft. spans, from the Ames experiments, and one of 78-ft. span by Professor Dufour, all with concrete floors, indicate that, when normal unit stresses are developed, the impact in truss members will be well below 25%, except when artificial obstructions are used. The data in Fig. 2 suggest the same for reasonably smooth timber floors. Fig. 1 has been plotted from the data in Table 2, Appendix A, and from Professor Dufour's experiments.[11] Fig. 2 has been plotted from the data in Table 3, Appendix A.

The results on trusses, however, should be considered more in respect to the absolute value of the impact increment than to the ratio of this to the static stress, as the impact increment is not likely to be much increased as the bridge becomes more fully loaded and the static stresses become greater.

---

[11] *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, pp. 1737–1742.

Under such fully loaded conditions the impact percentages will be much smaller than those indicated in Figs. 1 and 2 and will decrease rapidly with an increase in the span length.

Recommendations for Impact in Trusses of Highway Bridges.—Taking these matters into consideration, the Committee recommends for the girders and trusses of highway bridges:

1.—That for spans of less than 40 ft., the impact increment of stress be assumed as 25% of the live load stress, the same as for floors.

2.—That for spans of 40 ft. and more, the impact increment of stress be assumed as a fraction of the live load stress determined by the following formula:

$$I = \frac{50}{L + 160}$$

in which, I = impact fraction; and, L = span length, in feet.

BY-PRODUCTS OF THE INVESTIGATION

The stress measurements which were a necessary part of the investigations for impact, incidentally brought out information of interest and value in regard to the distribution of stresses throughout the trusses and the floor, as follows:

1.—In Table 4, page 13, *Public Roads* for September, 1924, a number of instances are cited where the unit stress in one part of a built member is more than double that in other parts of the same member. More instances of the same nature are given in *Bulletin 63* of the Engineering Experiment Station of Iowa State College.

2.—Table 2, page 11, *Public Roads* for September, 1924, shows that the unit stresses in the lower chords of a steel bridge with a concrete floor are far less, and those in the upper chords are slightly more, than the stresses computed under the usual assumptions of neglecting a possible lower chord tension in the floor. It is evident that the floor takes a considerable part of the lower chord tension, raises the plane of the lower chord stresses, reduces the effective depth of the truss, and thereby increases slightly the stress in the upper chords.

3.—In *Bulletin 75* of the Engineering Experiment Station of Iowa State College, there are eight plates showing the distribution of static stresses in the stringers of four bridges with concrete floors. They show, in all instances, that the stresses developed are much less than those computed on the assumption that the steel stringers alone support the load. They point to consistent T-beam action between the concrete and the steel.

SUGGESTIONS FOR FUTURE WORK

Uncertainties in the distribution of stress over the various parts of a truss member are much greater than uncertainties in impact, at least in the trusses of highway bridges.

It would seem, therefore, that further investigations for impact as useful information for a basis for design, should be accompanied, or perhaps preceded, by a general study of the distribution of stresses, including secondary stresses, stresses due to eccentric connections, etc. Such an investigation should necessarily be of sufficient magnitude to cover a wide range of conditions and to yield conclusions of general application. It would be a worthy companion of the research now under way through the American Society for Testing Materials for studying the distribution of stresses throughout the area of rolled sections. That research is based on the fact that the different parts of a rolled section were really different materials due to unequal working in the process of rolling. Its ultimate effect may be to improve the methods of rolling and to provide material of greater uniformity.

The bridge truss research would give information concerning the degree to which stress is transmitted from one composite part of a riveted (or a welded) member to another. This might possibly lead to better fabrication as well as point out the parts of a structure where the stress distribution is least uniform. In either event, it would contribute to the understanding of some of the factors that must be considered in the determination of unit stresses, a problem which was considered by the Society a few years ago, but regarding which no satisfactory solution was reached.

The bridge truss research should disclose whether there is any need for further work on impact in trusses of highway bridges, and it also might point the way toward the best procedure for the impact investigations if they were found to be desirable.

Respectfully submitted,

Special Committee on Impact in Highway Bridges,

A. H. Fuller, *Chairman*,
Arthur R. Eitzen,
E. F. Kelley,
F. E. Turneaure.

December 5, 1928.

APPENDIX A.

Stress and Force Measurements for Impact on Bridges

In order that those who do not have access to *Bulletin 73* of the Engineering Experiment Station of Iowa State College, mentioned in the report, may get a general idea of the manner in which the data of that *Bulletin* were gathered and recorded, a brief description of these factors is given.

The bridges most used were: (*a*) A 150-ft. through riveted steel span, with 6-in. concrete floor on nine lines of 10-in., 25-lb. steel stringers of 18 ft., 9-in. span; (*b*) a 33-ft. I-beam span, with 6-in. concrete floor on nine lines of 15-in., 43-lb. steel stringers; (*c*) a 26-ft. I-beam span, with 8-in. concrete floor on nine lines of 18-in., 54.7-lb. steel stringers; and (*d*) seven light steel spans, with timber floors on steel stringers.

The loads consisted of 3½-ton Liberty trucks with total loads varying from 5 tons for the empty truck to 15 tons, with maximum rear axle load of 12 tons. The unsprung weight of one rear axle was 4 400 lb.

The instruments most used for stress measurements were the McCollum-Peters electric telemeter and the West extensometer.[12]

Impact in Floors.—The force of the blows of the truck wheels on the bridge floors was determined by accelerometers which were designed and constructed for the work and described in *Bulletin 75* of the Iowa Engineering Experiment Station. These accelerometers were similar in design and action to those used by the U. S. Bureau of Public Roads.[13]

In the preparation of the *Bulletin* from which these conclusions were taken, the stress measurements were platted on a large number of diagrams, and median lines were drawn. All these lines were then assembled, for a certain stringer (but without the platted points), on a single diagram, as shown on Fig. 4,[14] and other similar diagrams. A study of Fig. 4 brings out certain facts, within the limits of those experiments:

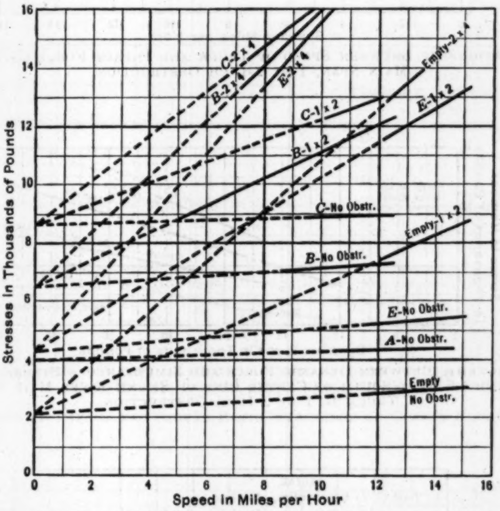


FIG. 4.—RELATIONSHIP BETWEEN SPEED OF TRUCK AND STRESS IN STRINGERS. SIX-INCH CONCRETE FLOORS, SKUNK RIVER MAIN SPAN, STRINGER 7.5 FEET SOUTH OF CENTER LINE.

1.—The increase in stress varies directly with the speed.

2.—The stress increases but slightly with the speed on clean floors (no obstructions).

3.—The stress increases decidedly with the speed when obstructions are used.

4.—The increase in stress, for a given obstruction and speed, is approximately the same for all trucks. This indicates that the impact increment of stress is caused primarily by the unsprung weight of the truck.

---

[12] *Proceedings*, Am. Soc. C. E., March, 1923, Papers and Discussions, p. 460.

[13] *Public Roads*, December, 1924, pp. 1–9.

[14] *Bulletin 75*, Eng. Experiment Station, Iowa State Coll., Fig. 14.

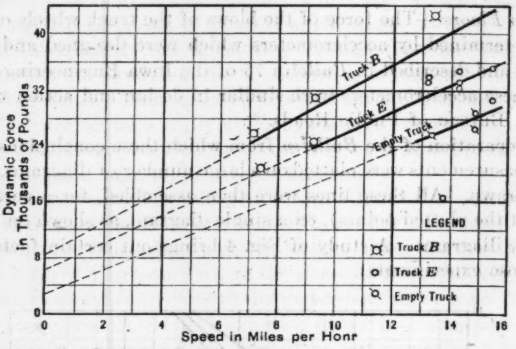


FIG. 5.—RELATIONSHIP BETWEEN SPEED OF TRUCK AND IMPACT FORCE ON SKUNK RIVER MAIN SPAN, 1 BY 2-INCH OBSTRUCTION.

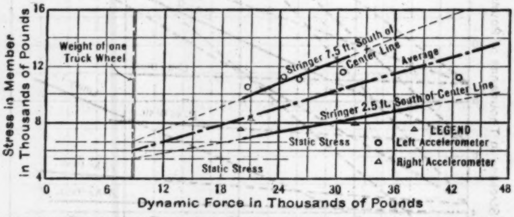


FIG. 6.—RELATIONSHIP BETWEEN DYNAMIC FORCE AND SIMULTANEOUS STRESSES IN STRINGERS 2.5 AND 1.5 FEET SOUTH OF CENTER LINE OF SKUNK RIVER MAIN SPAN, TRUCK B, 1 BY 2-INCH OBSTRUCTION.

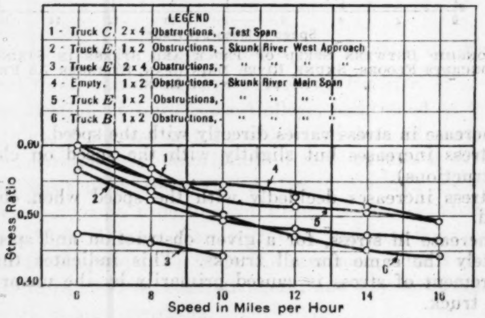


FIG. 7.—RELATIONSHIP BETWEEN SPEED OF TRUCKS AND STRESS RATIO FOR STRINGERS OF CONCRETE FLOOR BRIDGES.

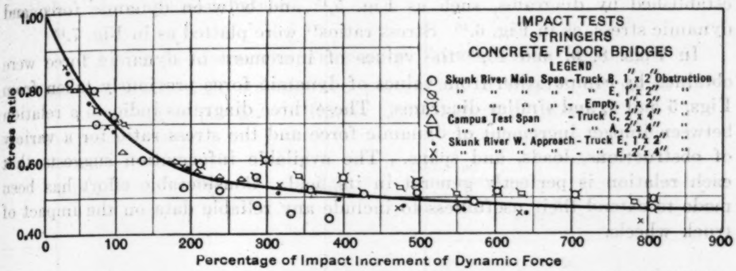


FIG. 8.—RELATION OF IMPACT AND STRESS RATIO, STRINGERS OF CONCRETE FLOOR BRIDGES.

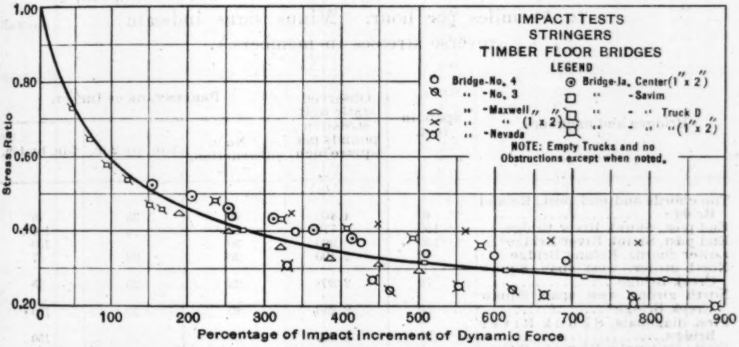


FIG. 9.—RELATION OF IMPACT AND STRESS RATIO, STRINGERS OF TIMBER FLOOR BRIDGES.

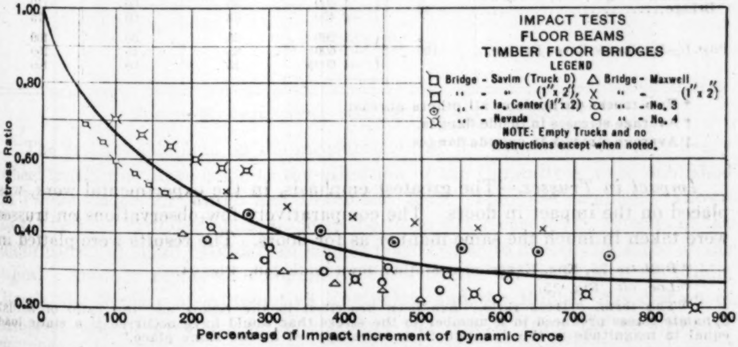


FIG. 10.—RELATION OF IMPACT AND STRESS RATIO, FLOOR-BEAMS OF TIMBER FLOOR BRIDGES.

A relationship between speed and the dynamic force of a wheel blow was established by diagrams, such as Fig. 5,[15] and between dynamic force and dynamic stress, as in Fig. 6.[16] Stress ratios[17] were platted as in Fig. 7.[18]

In Figs. 8, 9, and 10,[19] the values of increment of dynamic force were obtained by computation from values of dynamic force previously taken from Figs. 5 and 6, and similar diagrams. These three diagrams indicate a relation between impact increment of dynamic force and the stress ratio for a variety of obstructions, loads, and spans. The available information suggests that each relation is perfectly general in its field. Considerable effort has been made to extend their usefulness to include any reliable data on the impact of truck wheels.

TABLE 2.—IMPACT IN TRUSSES OF BRIDGES WITH CONCRETE FLOOR.

(Observed static unit stresses and percentage of impact in Skunk River, Squaw Creek, and Roland Bridges. Two 14-ton trucks; speed, 10 to 14 miles per hour. Minus signs indicate reverse stresses in members.)

Bridges and members.	Span, in feet.	Observed static unit stress, in pounds per square inch.	PERCENTAGE OF IMPACT.		
			No obstruction.	1-in. by 2-in.	2-in. by 4-in.
Top chords and end post, Roland Bridge	40	4 400	20	35	78
End post, Skunk River Bridge	150	770	30	..	100
End post, Skunk River Bridge*	150	920	30	..	100
Lower chords, Roland Bridge	40	2 400	20	33	67
North girder, west span, Squaw Creek Bridge	70	2 300	20	25	75
North girder, west span, Squaw Creek Bridge*	70	2 250	25	30	100
First diagonals, Skunk River Bridge	150	2 000	50	..	150
First diagonals, Skunk River Bridge*	150	2 300	30	75	100
First diagonals, Roland Bridge	40	5 200	30	40	100
Remaining diagonals, Skunk River Bridge	150	3 200	20	30	75
Remaining diagonals, Roland Bridge	40	1 600 −2 400	35 33	60 60	110 75
Post U_2-L_2, Skunk River Bridge	150	1 200 −1 600† −1 600‡	25 25 15	30 45 30	100 150 100

* Two trucks in tandem, all others abreast.

† Average stresses in inside flanges.

‡ Average stresses in outside flanges.

Impact in Trusses.—The greatest emphasis, in the experimental work, was placed on the impact in floors. The comparatively few observations on trusses were taken in much the same manner as for floors. The results were platted in form similar to those for floors. Percentages of impact increment were taken from the diagrams and tabulated.

---

[15] *Bulletin 75,* Eng. Experiment Station, Iowa State Coll., Fig. 24.

[16] *Loc. cit.,* Fig. 25.

[17] The term, "stress ratio," has been described in the report as "the ratio of actual dynamic stress produced in a member to the stress that would have occurred if a static load equal in magnitude to the dynamic force were applied at the same place."

[18] *Bulletin 75,* Eng. Experiment Station, Iowa State Coll., Fig. 26.

[19] *Loc. cit.,* Figs. 29, 30, and 31.

TABLE 3.—IMPACT IN TRUSSES OF TIMBER FLOOR BRIDGES.

(Observed static unit stresses and percentage of impact for speed of 10 miles per hour. Empty truck except as noted.)

Bridges.	Span, in feet.	Member.	Observed static unit stress, in pounds.	PERCENTAGE OF IMPACT.	
				No obstruction.	1-in. by 2-in.
Nevada	80	L_0-L_1	740	90	...
		U_1-L_1	1 050	160	...
Iowa Center	79	L_0-L_1	2 500*	100	145
		L_1-L_2	850	55	205
		U_1-L_2	2 600	60	...
		U_2-L_2	2 750*	45	...
Maxwell	95	L_0-L_1	1 700	105	315
		L_1-L_2	500	250	540
		U_1-L_1	1 000	90	255
		U_2-L_2	2 200*	30	160
No. 3	79	L_0-L_1	700	390	...
		L_1-L_2	1 400	215	...
		U_1-U_2	2 300	90	...
		U_1-L_3	1 900	225	...
No 4	48	L_0-L_2	400	300	...
		L_2-L_3	1 900	125	...
		U_1-U_2	1 250	55	...
		U_1-L_2	2 500	185	...
		U_2-L_3	1 850	95	...
Savim	70	L_0-L_1	500	190	...
		L_1-L_2	1 800	145	290*
		U_1-U_2	1 700†	60	130
		U_1-L_1	5 500†	25	50
			4 700†	30	...
		U_2-L_1	3 500†	60	65

* Instrument on one bar only.

† Truck D.

Table 2[20] gives data for the trusses of bridges with concrete floors, and Table 3,[21] for trusses of bridges with timber floors. The results from Table 2 have been plotted in Fig. 1 and those from Table 3, in Fig. 2.

APPENDIX B

FORCE MEASUREMENT FOR IMPACT ON PAVEMENT

The first reliable data on the measurement of the dynamic force of a heavy wheel truck, that have come to the attention of the Committee, were published in *Public Roads* for March and December, 1921. They are commonly known as the "copper cylinder" data, because the force was measured by observing the deformation of small cylinders of copper placed so as to absorb the force of the wheel. Another report of co-operative tests by the U. S. Bureau of Public Roads, the Society of Automotive Engineers, and the Rubber Association of America was of great value in checking the results from the co-operative project at Ames, Iowa, and in interpreting the "copper cylinder" impact data.[22] Certain diagrams from the report are reproduced as a means of supplementing and interpreting the data which were obtained from other sources.

---

[20] Based on Table 4, *Public Roads*, September, 1924, p. 13.

[21] Based on Table 8, *Bulletin 75*, Eng. Experiment Station, Iowa State Coll.

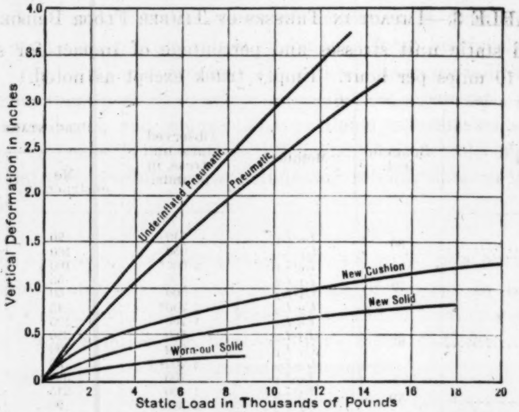


FIG. 11.—TYPICAL CURVES SHOWING TIRE ACTION UNDER STATIC LOAD.

The curves in Fig. 11[23] give some information concerning the flexibility of tires. The curve for "worn out" solid tires indicates a deformation of about 0.25 in. for a load of 10 000 lb. This is slightly softer than the tires used at Ames, Iowa, which had deformations of about 0.19 in. for the same load.

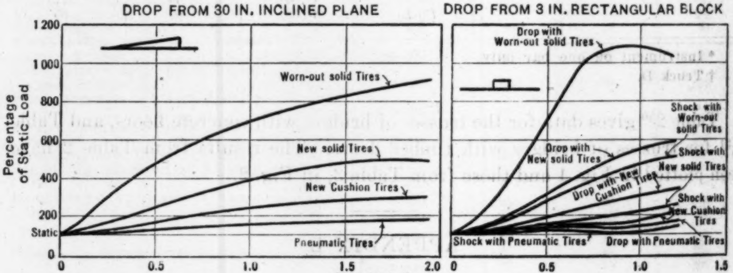


FIG. 12.—EFFECT OF HEIGHT AND TYPE OF OBSTRUCTION ON VERTICAL IMPACT REACTION.

Several results from impact tests are shown in Figs. 12, 13, and 14, all obtained with a 2-ton truck equipped with rated-size dual tires. In Figs. 12 and 13 the wheel loads are those of the tire capacities, and in Fig. 14 the influence of variable load is shown. The speed used in the tests of Figs. 12 and 14 was 12 miles per hour; those of Fig. 13 were variable. Fig. 12[24] shows about the same range of dynamic force of wheel blows as Figs. 8, 9, and 10, and, at the same time, indicates the variation in the intensity of the blows for different tires and obstructions.

---

[22] "Motor Truck Impact as Affected by Tires, Other Truck Factors, and Road Roughness," by James A. Buchanan and J. W. Reid, *Public Roads*, June, 1926.

[23] *Loc. cit.*, Fig. 2.

[24] *Loc. cit.*, Fig. 14.

Fig. 13[25] shows the relation between the speed and the force of the wheel blow for various tires and obstructions. In comparing these relations with those of *Bulletin 75*, Engineering Experiment Station, Iowa State College, some significant facts are brought out in the form of the curves. A straight line relation between the speed and the blow is indicated in the *Bulletin* for the observed speeds, which were from 5 to 15 miles per hour. The same relationship is generally indicated in Fig. 13 for the same speeds, but the latter shows a deviation from a straight line for low speeds (which is not important) and for speeds greater than 15 miles per hour. The data in Fig. 13 make an important contribution in indicating very little, if any, increase in impact for speeds greater than 15 miles per hour, within the limits of known experiments. A tendency toward critical speed at about 15 miles per hour was also indicated in one other instance.[26]

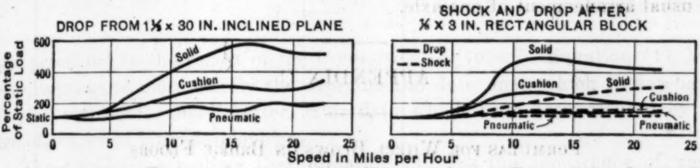


FIG. 13.—EFFECT OF TYPE OF OBSTRUCTION ON VERTICAL IMPACT REACTION AS INFLUENCED BY TRUCK SPEED.

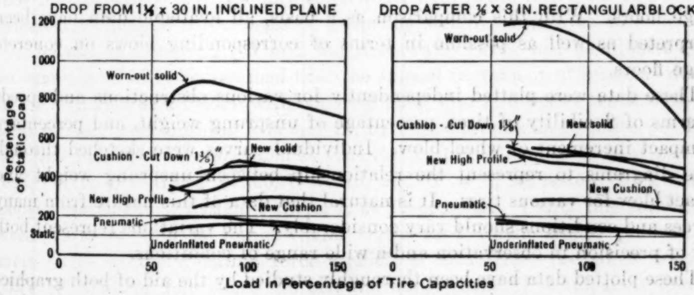


FIG. 14.—EFFECT OF LOAD IN TERMS OF TIRE CAPACITY ON VERTICAL IMPACT REACTION WITH SEVERAL TYPES OF TIRES AND IMPACT.

In Fig. 14[27] is indicated a decreased percentage of impact increment for increased tire loads for various types of tires. This factor is typical of two others mentioned elsewhere in this report; (*a*) That the percentage of impact decreases with an increase in the sprung weight; and (*b*) that the percentage of impact decreases with the flexibility of a structure, or as the unit stresses increase. These two factors suggest that the danger to a structure from an overload on a rubber-tired truck may not be as great as observed values would indicate; also, that for normally stressed and slightly overstressed structures, the impact factors will be less than those observed in the known experiments on bridges.

---

[25] "Motor Truck Impact as Affected by Tires, Other Truck Factors, and Road Roughness," by James A. Buchanan and J. W. Reid, *Public Roads*, June, 1926, Fig. 15.

[26] *Public Roads*, August, 1928, Figs. 10, 12, and 13, p. 122.

[27] "Motor Truck Impact as Affected by Tires, Other Truck Factors, and Road Roughness," by James A. Buchanan and J. W. Reid, *Public Roads*, June, 1926, Fig. 16.

Other interesting and pertinent factors were brought out—that impact is perceptibly greater when dual, rather than single, tires are used and decidedly less when the rear load is carried on two axles (four wheels) rather than on the usual arrangement of one axle.

APPENDIX C

Formulas for Wheel Blows on Bridge Floors

As has been stated in the report, the force of a blow from a given wheel depends on the rigidity of the body that is struck, as well as on the weight and the vertical velocity of the wheel and on the kind and condition of the tire. The blow of a moving wheel on a bridge floor is less than the blow of the same wheel on a pavement, whenever the flexibility of the structure is greater than that of the pavement. This condition exists in most bridges, especially where the load is sufficiently great to produce deflections and unit stresses corresponding to fully loaded conditions.

The data from the experiments of the U. S. Bureau of Public Roads on pavements have been compared with corresponding data on the more flexible bridge floors. With this comparison as a basis, all available data have been interpreted as well as possible in terms of corresponding blows on concrete bridge floors.

These data were platted independently for various obstructions and speeds in terms of flexibility of tires, percentage of unsprung weight, and percentage of impact increment of wheel blow. Individual curves were sketched through these diagrams to represent the relationship between unsprung weight and impact blow for various tires. It is natural that data of this nature from many sources and conditions should vary considerably. The variations represent both lack of precision in observation and a wide range of conditions.

These plotted data have been thoroughly studied by the aid of both graphics and mathematics, and it has been found impossible to evolve a simple equation which will closely represent the experiments. The formula,

$$I = \left(\frac{p\,H^{0.45}\,S^{0.54}}{15\,d^{0.3}}\right)^{a} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

does represent them and is presented as an equation which best sums up the available knowledge of the impact of wheel blows of present-day heavy trucks on the surface of concrete floors which rest on steel stringers. In this equation:

I = impact increment of dynamic force of wheel blow expressed as a percentage of the static load.

p = unsprung weight expressed as a percentage of total weight of truck.

a = numerical exponent having the value, $\dfrac{1}{0.22\,p^{0.2}}$.

H = height, in inches, of any obstruction on the bridge floor. (Equals about 0.1 in. for the bare concrete floors and an average of the best timber floors for which data are available.)
S = speed of truck, in miles per hour.
d = tire deformation, in inches, under a static load of 10 000 lb.

A simpler equation,

$$I = \frac{1.8\, H\, S\, p^{0.625}}{d^{0.45}}$$

was presented in the report of the Committee for 1926.[28] Equation (1), however, represents more closely the experimental data upon which it was based. It also represents, much better, later data which have been secured by the use of trucks of later development.

The Committee realizes that Equation (1) has the appearance of undue precision and that it is cumbersome. Numerous simpler ones have been evolved, but all are complicated and none represents, nearly as well, the results of experiments, especially upon the heaviest trucks; and the heavy trucks are really the only ones that are important in design.

Equation (1) gives directly, not the stress impact, but the impact of the blow of the truck wheel on a concrete bridge floor. In order to transform the effect of the blow into stress impact, it is necessary to make use of the stress-ratio curve for concrete floor in Appendix A (see Fig. 8). Equation (1) and the stress-ratio curve have been used in computing Table 1. It is believed they may be safely used for any conditions, within the limits of Table 1 and extended to include percentages of unsprung weight as high as 40.

The use of Equation (1) and the stress-ratio curve will be illustrated by an example. Let it be assumed that the impact increment of stress is desired in the stringers of a bridge with concrete floor, due to a heavy truck with new solid rubber tires, running over a 1 in. by 2-in. obstacle at 15 miles per hour, and that the truck is overloaded so that the unsprung load is 15% of the total load. With $p = 15$, $d = 0.6$, $S = 15$, and $H = 1$, Equation (1) gives the impact increment of dynamic force, I, as 72 per cent. Using this in Fig. 8 a stress ratio of 0.76 is found. The static load which would produce the same stress as the dynamic force (which is 1.72 times the wheel load) is (0.76×1.72), or 1.30 times the original static load. The impact increment of stress is, therefore, $1.30 - 1.00 = 0.30$, or 30 per cent.

APPENDIX D

RELATION BETWEEN BLOW UPON PAVEMENT AND UPON A BRIDGE FLOOR

Many data are available, mostly through the U. S. Bureau of Public Roads, which give the intensity of a wheel blow, and, therefore, the impact, on pavements. This information may be made available for determining the impact on bridges by establishing a relationship between the intensity of blows on pavement and on bridges under conditions which otherwise are similar.

---

[28] *Proceedings*, Am. Soc. C. E., March, 1926, Papers and Discussions, p. 448.

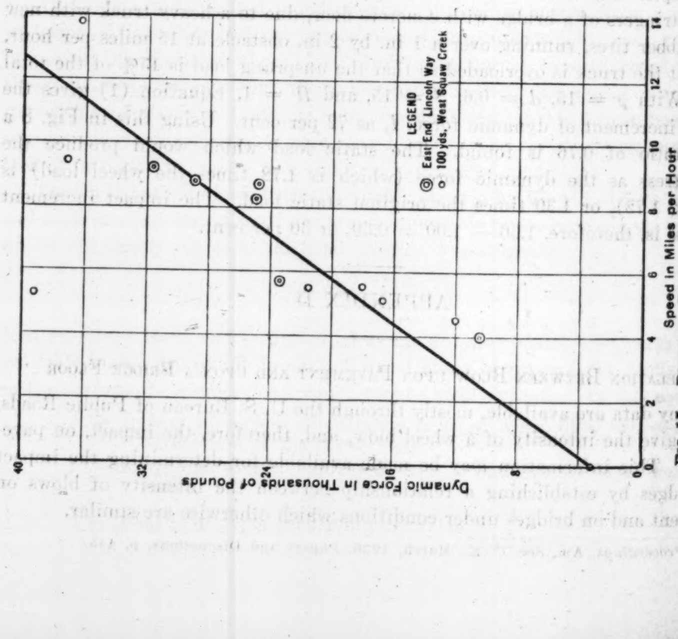


FIG. 15.—DYNAMIC FORCE OF WHEEL BLOWS ON CONCRETE PAVEMENT, EMPTY LIBERTY TRUCK, RIGHT ACCELEROMETER, 1-INCH BY 2-INCH OBSTRUCTION.

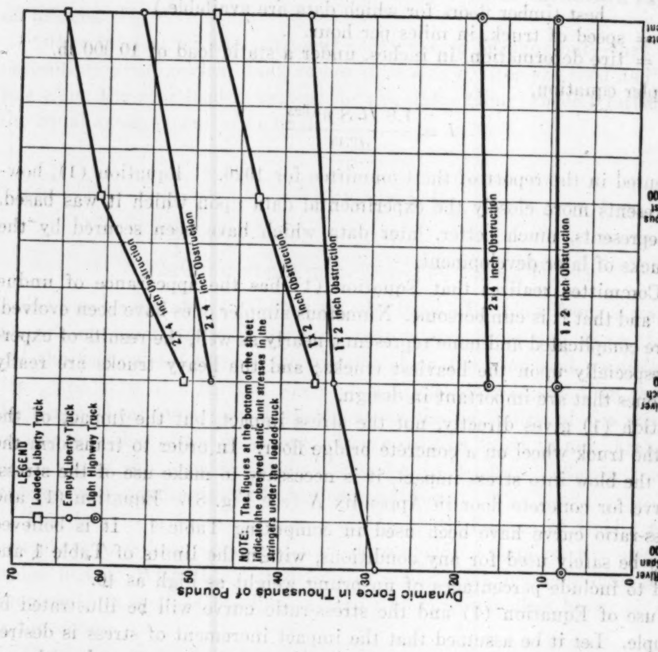


FIG. 16.—COMPARISON OF DYNAMIC FORCE OF WHEEL BLOWS ON PAVEMENTS AND ON BRIDGES.

A number of experiments not only verify the well-known fact that the intensity of the blow from any wheel is dependent on the flexibility of the surface on which the blow is delivered, but suggest a value for the relationship, fairly well in keeping with the variation in road surfaces, tires, etc.

TABLE 4.—DYNAMIC FORCE OF WHEEL BLOWS, IN THOUSANDS OF POUNDS, ON PAVEMENT AND ON BRIDGES.

(Speed, 12 miles per hour.)

	Obstruction.	LIBERTY, LOADED.			LIBERTY, EMPTY.			LIGHT HIGHWAY TRUCK.		
		Accelerometer.			Accelerometer.			Accelerometer.		
		Left.	Right.	Average.	Left.	Right.	Average.	Left.	Right.	Average.
Concrete pavement	1-in.	48.0	......	......	37.2	38.0	37.6	......	......	9.8
Skunk River main span	1-in.	......	......	......	28.5	28.5	28.5	......	......	8.3
Skunk River approach	1-in.	36.0	36.0	36.0	34.0	34.4	34.2	......	......	8.8
Campus test	1-in.	42.4	44.0	43.2	......	......	......	......	......	......
Concrete pavement	2-in.	64.0	65.2	64.6	57.0	56.0	56.5	......	......	18.3
Skunk River approach	2-in.	51.0	......	......	47.8	48.4	48.1	......	......	16.2
Campus test	2-in.	61.0	60.8	60.9	......	......	......	......	......	......
Static wheel load, in pounds	.....		8 800			3 350			1 350	
Unsprung wheel load, in pounds	.....		2 200			2 200			1 000	

Unit stresses, deflection, and impact in the stringers of three bridges of various degrees of flexibility, are given in Table 6 of *Bulletin 75* of the Iowa Engineering Experiment Station. Although these data do not justify a curve

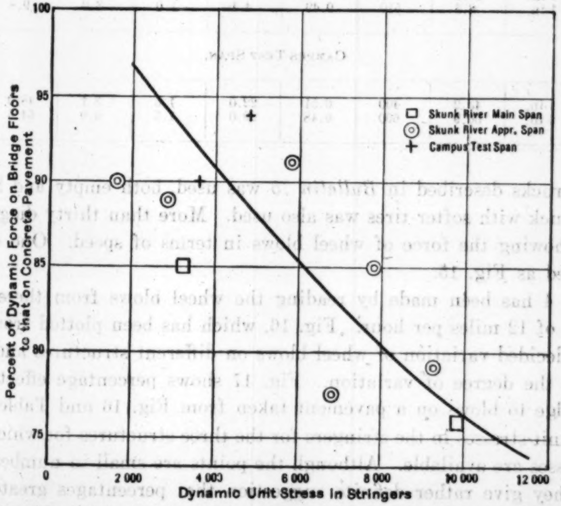


FIG. 17.—RELATION BETWEEN DYNAMIC FORCE ON CONCRETE FLOOR OF BRIDGES AND ON CONCRETE PAVEMENT IN TERMS OF DYNAMIC UNIT STRESS IN STRINGER.

or an equation, they do indicate much greater impact for the stiffer structures with small unit stresses and small deflection.

In the summer of 1927, a limited number of runs were made by the Engineering Experiment Station at Iowa State College in which the force of a wheel blow on pavements and on bridges was determined. One of the 3½-ton Liberty trucks described in *Bulletin 75* was used, both empty and loaded. A lighter truck with softer tires was also used. More than thirty diagrams were plotted showing the force of wheel blows in terms of speed. One of these is reproduced as Fig. 15.

TABLE 5.—DYNAMIC UNIT STRESSES AND RATIO OF WHEEL BLOWS ON THREE BRIDGES WITH CONCRETE FLOOR AND ON CONCRETE PAVEMENT.

(Data from Table 4; speed, 12 miles per hour; loads and forces, in thousands of pounds.)

Static load.	Obstruction.	Dynamic force.	Percentage impact.	Stress ratio.	Equivalent static force.	Static unit stress.	Dynamic unit stress.	Blow on pavement.	Ratio, Blow on bridge / Blow on pavement
				SKUNK RIVER APPROACH SPAN.					
8.8	1-in.	36.0	310	0.52	18.7	3.2	6.8	48.0	0.75
8.8	2-in.	51.0	480	0.49	25.0	3.2	9.1	64.6	0.79
3.35	1-in.	34.2	920	0.47	16.0	1.2	5.8	37.6	0.91
3.35	2-in.	48.1	1 340	0.45	21.6	1.2	7.8	56.5	0.85
1.35	1-in.	8.8	550	0.48	4.2	0.5	1.6	9.8	0.90
1.35	2-in.	16.2	1 100	0.46	7.5	0.5	2.8	18.3	0.89
				SKUNK RIVER MAIN SPAN.					
3.35	1-in.	28.5	750	0.48	13.7	2.4	9.8	37.6	0.76
1.35	1-in.	8.3	510	0.49	4.1	1.0	3.0	9.8	0.85
				CAMPUS TEST SPAN.					
8.8	1-in.	43.2	400	0.51	22.0	1.5	3.7	48.0	0.90
8.8	2-in.	60.9	600	0.48	29.0	1.5	4.9	64.6	0.94

Table 4 has been made by reading the wheel blows from these diagrams at speeds of 12 miles per hour. Fig. 16, which has been plotted from Table 4, shows a decided variation of wheel blows on different structures and gives the basis for the degree of variation. Fig. 17 shows percentage effects of blows on a bridge to blows on a pavement taken from Fig. 16 and Table 4, plotted against unit stresses in the stringers for the three structures for which observed unit stresses are available. Although the points are small in number and scattering, they give rather definite suggestion that percentages greater than 80 are for such small unit stresses that they would not be significant for design purposes. The necessary computations for plotting Fig. 17 are given in Table 5.

It is impossible to draw precise conclusions from data such as these; neither would precise conclusions be warranted from any experimental or theoretical studies. The inherent variations in structures and vehicles preclude results of high precision. On the other hand, it seems apparent:

First.—That the impact is less on a bridge than on the rigid types of pavements; and,

Second.—That the intensity of a blow from a truck wheel on a bridge floor, when the unit stresses in stringers or floor-beams approximate design values, may approach, but will not exceed, 80% of the blow of the same wheel on a reasonably rigid pavement.

DISCUSSION

Searcy B. Slack,[29] M. Am. Soc. C. E. (by letter).—In compiling and considering available information on impact stresses in highway bridges the Committee has rendered a distinct service to all engineers concerned. The allowance made for impact in highway bridge design has varied widely, being different in almost every specification. Some engineers have taken the position that the available data were not conclusive and have made no allowance for impact, while others, through probable misinterpretation of the data, have made allowances which appear unreasonably high. The recommendation of the Committee is a happy medium between these extreme positions.

One of the main criticisms which will undoubtedly be made of the report is that the impact allowance for a given span length is the same for all truss members, whereas maximum stresses in the web members are caused by much shorter loads than the full span length; hence, greater impact allowance should be made. To meet this condition, L of the impact formula has frequently been taken as the loaded length instead of the span length, as suggested by the Committee. The result of taking L as the loaded length is that the allowance made for impact is uniform for the chord members, but varies for the web member in each panel. From the meager test data available this seems to be a needless refinement and the Committee is to be commended for simplifying the practice by recommending a formula which will give a uniform allowance throughout a given span. A further justification of this course is that the sheer weight of a modern highway bridge with concrete floor and paving has a great absorbing inertia, and a large part of the stresses in the members is due to dead load (except of course counters). Varying the impact allowance has comparatively little effect on the members.

In certain cases such as counters or unusual web designs, the formula proposed by the Committee probably does not provide sufficient allowance for impact. These are such special and limited cases, however, that a general formula could hardly be expected to apply to them.

In the interest of uniformity and simplification of highway bridge specifications it is to be hoped that the recommendations of the Committee will be widely adopted.

Llewellyn N. Edwards,[30] M. Am. Soc. C. E. (by letter).—This report contains instructive and valuable information. However, it seems unfortunate that the research tests and the data secured therefrom, which have been studied by the Committee, did not cover a wider range of highway bridge types, span lengths, and conditions of loading. A large proportion of American highway bridges are short-span structures, or are composed of multiple spans of short length. Incidentally, a considerable percentage of the superstructures of these bridges are constructed of reinforced concrete and are of either the slab or the T-beam type. In general, reinforced concrete slabs are used for spans varying from 2 ft. to 20 ft. in length and T-beams from

---

[29] Bridge Engr., State Highway Board, Atlanta, Ga.

[30] Care, Div. of Tests, U. S. Bureau of Public Roads, Washington, D. C.

21 ft. to 45 ft., with occasional structures ranging up to 60 ft., in length. The reinforced concrete through-girder type of superstructure is used to a limited extent. The test data studied by the Committee contain little information that can be construed to apply to these structures.

It is mentioned in the report that, apart from the physical irregularities involved in roadway floor surfaces, the design of the structure itself was found to be a factor involved in the determination of impact stresses. From examinations of the physical condition of many bridge structures, the writer has been forced to recognize that not only adequacy in the proportioning of the sections of the main members of bridges, but also an equal liberality in the proportioning of the detail parts forming the connections between the individual members, is of paramount importance in the service functioning of these structures. Until more is known definitely about the ways in which different types of bridge structures and their details of construction function under loadings, it seems hardly proper to predicate impact allowances upon other than a basis that applies to all types of highway bridges. Designers cannot assume with safety that the vehicles which pass over any structure will be equipped with wheels of a kind likely to produce minimum impact effects, nor can they assume that the materials or the methods of construction used will produce structures of an assumed good practice standard, or better. Irregularities in the surfaces of bridge roadways may result from poor workmanship incidental to their original construction, but there are perhaps more instances wherein failures of the surface resulting from traffic wear and tear have produced real obstructions to vehicle movement equal in magnitude and in many cases greater than the 1-in. by 2-in. obstruction used in the bridge impact test. However, apart from these conditions, it is fairly common, in areas where small stones and pebbles abound, to find these obstructions scattered promiscuously upon the surface of the roadway.

For motor trucks and other vehicles provided with springs, the impact effort is evidently produced by the unsprung load and the intensity of the spring pressure; but for other wheeled loads, as, for example, heavy road rollers, shovels, and other heavy portable equipment, there is no sprung load factor involved. In general, the speed rate of movement of these loadings of the latter class is comparatively slow, but frequently the loads are moved by truck traction rather than by their own power and in such cases speed may become a factor.

The Committee recommends "that for the design of highway bridge floors and floor-beam suspenders, the impact increment of stress be assumed as 25% of the live load stress", and suggests that this percentage should be used only when the floors are sufficiently smooth to conform to good modern practice. The designer can have no way of knowing just what will be the character of the roadway surface produced on the bridge for which he, perhaps, does not even prepare the construction detail plans.

The curves shown in Fig. 3 of the report are perhaps misleading in relation to the Conference Committee (A. R. E. A. and A. A. S. H. O., 1927) impact formula since this formula applies to all span lengths, instead of only to those having effective lengths of 40 ft. and more. In speculations and meditations upon this problem of impact the writer has held rather rigidly to the idea that, in bridge structures, length of span—as a factor tending to provide a means of diffusing, distributing, and absorbing its dynamic effects—is an element to be considered. With this idea in mind he has assumed, naturally enough, the fundamental condition to be that of the impact effect of traffic on a pavement as applying practically without diminution to bridges of short span—in the culvert class—wherein the superstructure is commonly either built monolithically with the substructure, or is very rigidly supported thereon. This seems to be a logical assumption and is quite evidently the basis of the formula promulgated by the Conference Committee. Incidentally, the initial 40% impact increment chosen by the Committee seems to be justified by the tests.

In the investigations of the U. S. Bureau of Public Roads, the tests made with the object of co-ordinating the results secured by the use of the impact machine with results obtained by using motor trucks developed a condition which may be regarded as pertinent to the question under discussion, since they show a relation between rate of speed and height of obstruction, or its equivalent in irregularities of roadway surface. These comparative tests show that, in general, the variations in impact reactions resulting from speeds varying from 4 to 12 miles per hour are practically equal to the variations resulting from obstructions ranging in height from 0.33 in. to 1.38 in. (⅜ in. to 1⅜ in.).

The impact test results studied by the Committee indicate rather sharply how much engineers should know concerning bridge structures, which has not yet been discovered by actual research tests and a study of the information so obtained.

IMPACT IN HIGHWAY BRIDGES COMMITTEE[32] (by letter).—The members of the Committee agree with Mr. Edwards that it is unfortunate that the data upon which the report was based, did not cover a wider range of conditions. However, they are more optimistic than he in believing that the data submitted are distinctly useful in anticipating the impact on reinforced concrete floors. The rigidity of a reinforced concrete floor is doubtless greater than that for a concrete floor on steel stringers and less than that for a concrete pavement. In Fig. 17, the Committee gives the relation between the dynamic force on bridge floors consisting of concrete floor on steel stringers and on concrete pavement. This diagram should be helpful in estimating the dynamic force on a reinforced concrete floor in terms of a known force on either a pavement or steel bridge floor.

The entire investigation shows a distinct reduction in impact with increase in unit stresses or, in other words, with increase in flexibility. Fig. 17 shows that the highest unit stresses observed were 10 000 lb. per sq. in., and indicates that with further increase in unit stresses, the impact would be reduced. The second conclusion of Appendix D of the Committee's report is so conservative

---

[31] *Public Roads*, Vol. 9, No. 6, August, 1928.

[32] A. H. Fuller, *Chairman*, Arthur R. Eltzen, E. F. Kelley, and F. G. Turneaure. This closing discussion is dated April 27, 1931.

that it may be reasonable to include the reinforced concrete floor in the general statement. For an unusually rigid concrete floor, it may be well to raise the 80% to 85 or 90%, or to another value that may meet the judgment of a designer or specification writer.

Mr. Edwards is correct in pointing out that the Conference Committee (A. R. E. A. and A. A. S. H. O., 1927) impact formula is shown incorrectly in Fig. 3, since the formula applies to all span lengths instead of only to those having span lengths of 40 ft., or more. The formula gives an impact increment of 40% for zero span rather than the 25% recommended by the Committee. He also states that the initial 40% impact increment chosen by the Conference Committee seems to be justified by the tests, and, further, that "designers cannot assume with safety that the vehicles which pass over any structure will be equipped with wheels of a kind likely to produce minimum impact effects, * * *".

Without inquiring as to the derivation of the Conference Committee formula, it is apparent that any opinion as to its superiority must be largely a matter of personal judgment since such experimental data as were available to the Conference Committee were also available to this Committee.

Having in mind the five basic considerations enumerated under "Recommendations for Impact in Floors of Highway Bridges" in the report, and the added fact that pneumatic tires are rapidly replacing other types on heavy trucks and buses, it is the judgment of the Committee that its recommendations provide adequate allowance for impact stresses.

There is an implied discussion of the report by the Society's Special Committee on Specifications for Bridge Design and Construction in the revision of Section 113[33] which was published in pamphlet form under date of June 27, 1929. The revised Section reads:

"*113.—Impact.*—The dynamic increment of the live load shall be added to the maximum computed live load stresses. For floor-beams and stringers, this increment shall be taken as 30%, and for floor-beam hangers as 60%, of the live load stress. For girders and truss members, the increment shall be determined by the formula:

$$I = S\left(\frac{50}{l + 160}\right)$$

in which,

I = impact or dynamic increment to be added to live load stresses;
S = computed maximum live load stress;
l = loaded length of a lane of traffic, in feet, producing the maximum stress in the member. For bridges carrying more than one lane of traffic, the aggregate loaded length of all lanes producing the stress, shall be used.

"Impact shall not be added to stresses produced by longitudinal and lateral forces."

The Section as revised differs from the recommendation of the Committee in two distinct points: First, it retains the previous provision of 30% for floor-beams and stringers and of 60% for floor-beam hangers, while the report of the Committee, based upon later information, recommends 25%

---

[33] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1914), p. 1279.

for floor-beams, stringers, and floor-beam hangers; and, second, the Specification Committee accepts the formula for trusses prepared by the Impact Committee, but retains the modified loaded length of the original specifications rather than accept the single-span length, as recommended by the Impact Committee. There is unquestionably some logic in the point of view of the Specification Committee. For practical use, however, the single length apparently represents just as well the available knowledge on the subject. It is surely much simpler to use. Mr. Slack, in his discussion, supports the view of the Impact Committee.

In a discussion of the 1926 progress report of the Committee, F. O. Dufour, M. Am. Soc. C. E., presented some material on impact in highway bridges under light loads observed by him from 1908 to 1912.[34] The results of this work, as well as of other work for light loads observed by the Committee,[35] have very little bearing on the design of modern highway bridges. However, these results are historically interesting and are doubtless correct for the conditions under which they were obtained.

Since the report of the Committee was presented in January, 1929, no additional work on the effect of impact in bridges has been brought to its attention. The U. S. Bureau of Public Roads, however, has been continuing the researches on impact on pavements. Two reports of this work have been published[36] by James A. Buchanan, Associate Engineer of Tests, Division of Tests, U. S. Bureau of Public Roads. One of these reports is entitled "Motor Truck Impact as Affected by Rubber Tread Thickness of Tires". The object of this particular phase of the impact investigation was to determine the effect of reducing the thickness of the tread rubber on the cushioning properties of typical solid and cushion tires. The second report was entitled, "The Inter-Related Effects of Load, Speed, Tires, and Road Roughness on Motor Truck Impact". The experimental work which forms the subject of this report was conducted on five different pavements, with a definition of the roughness of each. Various tires were used, from "pneumatic" to "worn solid", with wheel loads from 2 500 to 10 000 lb., and speeds up to 30 miles per hour. The report presents a number of curves showing the results for the conditions observed and gives directions for interpolating for other conditions.

For the purpose of comparison, the speeds at which each type of tire would cause the same impact reaction as that caused by the pneumatic tire at maximum speed, were determined; and Table 6 gives the recommendations for use.[37]

In both these reports the author states that since the observation of these tests, a thorough investigation of the accuracy of instruments used has been made by the U. S. Bureau of Standards, in co-operation with the U. S. Bureau of Public Roads. As a result of this investigation, a statement is made that the results obtained are, in general, systematically from 10 to 15% too low.

---

[34] *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1737–1742.

[35] *Public Roads*, September, 1924; parts of *Bulletins 63* and *75*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa.

[36] *Public Roads*, September, 1930.

[37] *Public Roads*, September, 1930, Table 9, p. 150.

The results reported by the Committee were taken from previous work by the Bureau of Public Roads and from other work upon which the accelerometers were calibrated by that Bureau. It seems likely, therefore, that the statement also applies to the report of the Committee.

TABLE 6.—SPEEDS AT WHICH APPROXIMATELY EQUAL IMPACT REACTIONS WOULD BE PRODUCED BY VARIOUS TYPES OF TIRE EQUIPMENT.

Tire equipment	Relative Speeds						
	In percentages	In miles per hour					
Pneumatic	100	40	35	30	25	20	15
New cushion	55	22	19	16	14	11	8
New solid	40	16	14	12	10	8	6
Worn solid	20	8	7	6	5	4	3

The reports mentioned give information which would be of great assistance in determining a basis for traffic regulation which would be applicable both on pavement and on bridges. If it is found practicable to determine the property of tires on various trucks and to maintain a speed regulation for each type of tire, or to eliminate the use of solid tires, much of the uncertainty of impact in bridges will have been removed.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1787

FLORIDA HURRICANE

FINAL REPORT OF THE COMMITTEE OF THE STRUCTURAL DIVISION[1]

WITH DISCUSSION BY MESSRS. F. O. DUFOUR, T. L. CONDRON, E. W. STERN, AND DAVID A. MOLITOR

In enumerating the effects of the Florida hurricane of September 17 and 18, 1926, it may be well to give a brief description of the storm. As in all tropical hurricanes, the wind blew in a circle with an area of calm in the center. F. L. Stearns, Assoc. M. Am. Soc. C. E., who visited Miami on behalf of the Committee soon after the storm, stated that the first half came from the northeast and the second half from a little east of south instead of from an exactly opposite direction. He felt that the explanation for this was that the storm changed form somewhat when it struck the land. He assumed that the center of the storm passed directly over Miami. It can be readily explained, however, by assuming that the center of the storm passed somewhat to the south of Miami. In Fig. 1 is shown a diagrammatic sketch of the storm. Point *A* represents the location of Miami during the first half and Point *B*, the location for the second half. The arrows indicate the direction of the wind. The time between the first half and the second half was about an hour, which indicates that Miami was not far removed from the path of the storm center.

In his report Mr. Stearns gave the storm effects on all the various forms of structures, but as a large part of these were not designed by engineers and are of such a character that they are not susceptible of close mathematical analysis, the Committee can add very little to present knowledge by a study of them. As the damage done to these structures was so many times greater than that done to structures designed by engineers, the necessity of technical design or more careful construction for the smaller structures as well as the larger ones is clearly shown.

It would be of considerable value to the Engineering Profession if an accurate determination of the wind pressure could be made, but the Committee does not deem it possible. On the top of the Meyer-Kiser Building was a small observation room, about 18 by 21 ft., with four small columns in the corners supporting the roof. The walls were blown out and the four columns bent. It would require a pressure of about 65 lb. per sq. ft. to produce the distortion which occurred and this figure is probably as good as can be determined; but it cannot be taken too seriously because the area of exposure at the time of failure is uncertain.

---

[1] Presented at the meeting of the Structural Division, New York, N. Y., January 19, 1928.

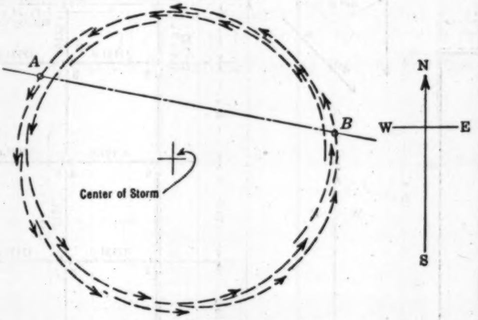
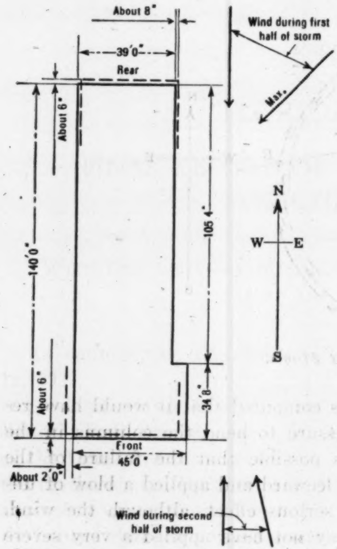


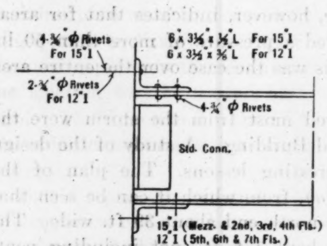
FIG. 1.—DIAGRAM OF STORM.

Professor Morris, of the Committee, has computed that it would have required about 60 lb. per sq. ft. of wind pressure to bend the columns in the front of the Meyer-Kiser Building. It is possible that the failure of the wall allowed the building to sag quickly to leeward and applied a blow of unknown force to the columns, producing a serious effect, although the wind, striking the building at an acute angle, may not have applied a very severe normal pressure. Hence, the static stresses computed may have little relation to the applied pressure. The evidence, however, indicates that for areas of moderate size, the storm probably exerted a pressure of more than 50 lb. per sq. ft., but there is no evidence that this was the case over the entire area of a building.

The two Miami buildings which suffered most from the storm were the Meyer-Kiser Building and the Realty Board Building. A study of the design of these two buildings teaches some interesting lessons. The plan of the Meyer-Kiser Building is shown in Fig. 2 (*a*), from which it can be seen that the south end is about 45 ft. wide and the north end about 39 ft. wide. The length is about 140 ft. The height was about 176 ft., not including penthouses. The dashed line shows the distortion from the storm. On Fig. 3 is shown a typical floor plan. The type of wind braces used is shown on Fig. 2 (*b*) and on Fig. 4. The connection shown is used at points marked *B* in Fig. 3. Based on the number and size of the rivets in the wind braces and assuming the elastic limit of the steel at 36 000 lb. per sq. in., the steel work alone, without any help from the walls, should have resisted a wind pressure of about 15 lb. per sq. ft. at the fifth floor. However, the clip angles were so thin that in bending they did not develop more than 25% of the value of the rivets. These clip angles in many cases bent or broke, thus destroying all the necessary strength; and even if they had not broken they would have been so limber as to have given the building little stiffness. The columns too were not suited to develop the strength for the type of connections used.


About 8"
39'0"
Rear
About 6"
140'0"
105'4"
34'8"
About 6"
Front
45'0"
About 2'0"
Wind during first half of storm
Max.
N
W
E
S
Wind during second half of storm

(a) DIAGRAM OF MEYER-KISER BUILDING


4-¾" φ Rivets For 15"I
6"x 3½"x ½"L For 15"I
6"x 3½"x ⅜"L For 12"I
2-¾" φ Rivets For 12"I
4-¾" φ Rivets
Std. Conn.
15"I (Mezz. & 2nd, 3rd, 4th Fls.)
12"I (5th, 6th & 7th Fls.)

(b) WIND BRACING DETAIL

FIG. 2.—PLAN AND DETAILS, MEYER-KISER BUILDING.

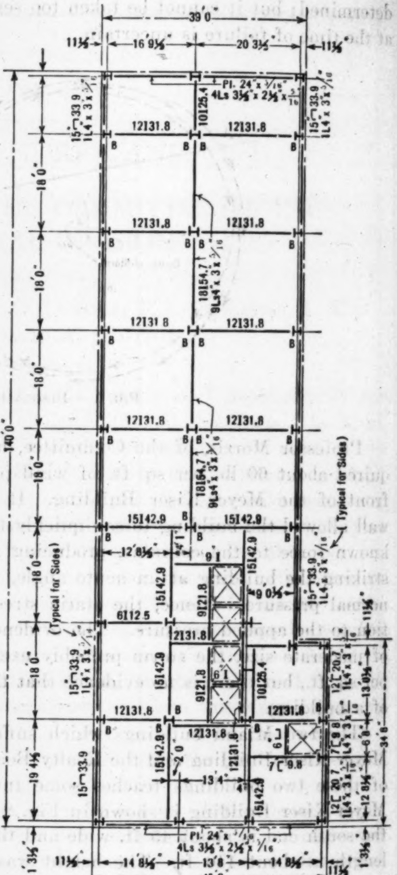

39'0"
11⅛
16'9⅛
20'3½
11⅛
Pl. 24"x ⁵⁄₁₆"
4Ls 3½"x 2½"x ⁵⁄₁₆"
15⌐33.9
12I31.8
10I25.4
B
18'0"
18I54.7
140'0"
15I42.9
9I21.8
6I12.5
12'8⅛
9'1
9'0⅛
(Typical for Sides)
13'4"
2'0
19'4"
7⌐9.8
12⌐20.7
45'0"
13'0⅝
12'8⅛
8'8
34'8
13½
14'8⅛
13'8
11⅛

FIG. 3.—TYPICAL FLOOR FRAMING PLAN, MEYER-KISER BUILDING.

The columns were made up of plate and angle sections and in the area that failed they had few cover-plates. The thickness of the angles in the columns was such that the outer rivets connecting the clips to the columns could not develop their full strength before bending the flange angles of the columns. In addition, in the south end of the building, the floor system was broken up with elevator shafts and there was very little bracing. What bracing there was, was on beams shallower than the 24-in. spandrel girders in the front or south wall. The result was that this wall took nearly all the wind load for this end of the building until it failed. At the north end there was bracing of a similar character, but more bracing in the floor system, which evidently is the reason for less distortion of this end of the building. In conclusion, it is perfectly evident that there was no adequate wind-bracing in this building, and the bracing which was used, was made valueless by the details. It is difficult to understand how any loans or hurricane insurance could have been obtained on this building. The results bring out forcibly the desirability of engineering advice for financial organizations as well as for the owners.

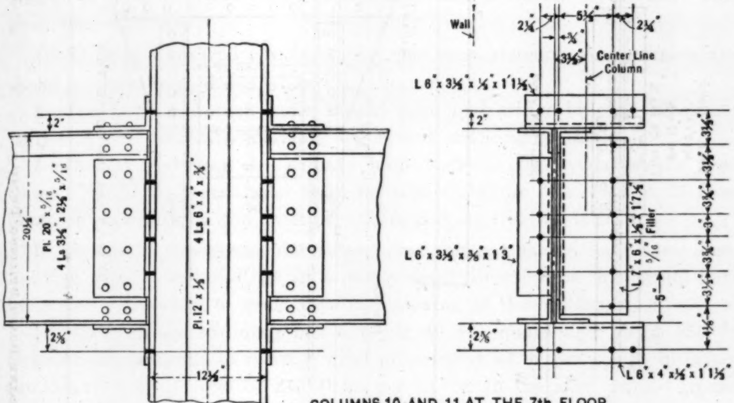
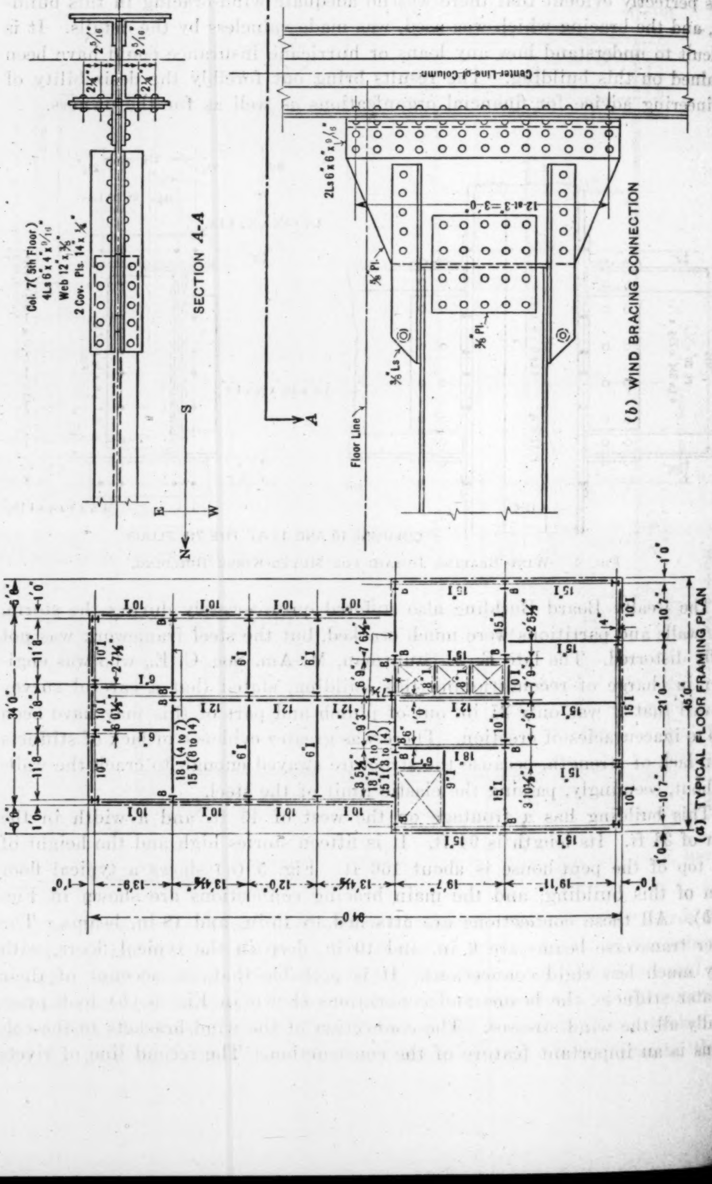


FIG. 4.—WIND-BRACING DETAIL FOR MEYER-KISER BUILDING.

The Realty Board Building also suffered quite severely during the storm. The walls and partitions were much cracked, but the steel framework was not badly distorted. The late E. A. Stuhrman, M. Am. Soc. C. E., who was engineer in charge of reconditioning this building, stated that a careful survey showed that it was only 3⅞ in. out of plumb and part of this may have been due to inaccuracies of erection. There was greater evidence of lack of stiffness than lack of strength, because the structure swayed enough to crack the walls without, seemingly, passing the elastic limit of the steel.

This building has a frontage on the west of 45 ft. and a width in the rear of 34 ft. Its length is 94 ft. It is fifteen stories high and the height of the top of the pent-house is about 166 ft. Fig. 5 (*a*) shows a typical floor plan of this building, and the main bracing connections are shown in Fig. 5 (*b*). All these connections are attached to 15-in. and 18-in. beams. The other transverse beams are 6 in. and 10 in. deep in the typical floors, with very much less rigid connections. It is probable that, on account of their greater stiffness, the beams and connections shown in Fig. 5 (*b*) took practically all the wind stresses. The connection of the wind brackets to the columns is an important feature of the construction. The second line of rivets


Col. 7 (5th Floor)
4Ls 6" x 4" x 9/16"
Web 12" x 3/8"
2 Cov. Pls. 14" x 3/4"
2 5/16"
2 1/4"
SECTION A-A
E
W
N
S
A
Center-Line of Column
2Ls 6" x 6" x 9/16"
12 at 3" = 3'-0"
3/8" Pl.
3/8" Ls
Floor Line
34'0"
11'8"
8'8"
1'0"
18 I (4 to 7)
15 I (8 to 14)
10 I
12 I
15 I
18 I
9 I
8
13'9"
13'4 1/2"
12'0"
19'7"
19'11"
94'0"
21'0"
10'7"

(a) TYPICAL FLOOR FRAMING PLAN

(b) WIND BRACING CONNECTION

FIG. 5.—REALTY BOARD BUILDING DETAILS.

connecting the brackets to the columns is practically valueless because the connection angles of the thickness used would be weak in bending at the root if the full strength of these rivets was developed. Mr. Stuhrman stated that the steel work was checked and found to comply with the Building Code which calls for a 20-lb. wind load. Probably the combined stresses used were 24 000 lb. per sq. in. Without entering into any controversy on this point, the Committee doubts whether the available strength (before a 24 000-lb. unit stress was reached in some parts) would withstand a full 20-lb. wind load. A study of the design, however, indicates that a genuine effort was made to brace this building properly and any shortcomings should not be attributed to an attempt at false economy. The following points, however, should have been given more attention:

1.—The two unbraced panels in the rear were elements of weakness and should not have been designed in this manner.

2.—The front and rear walls should have had stiffer bracing because a fairly stiff steel frame is necessary to prevent cracking of walls.

3.—Special provision should have been made at the stair opening near Column No. 14 to prevent the beam between Columns Nos. 14 and 15 from being bent laterally by the twisting of the rear end of the building.

In addition to these considerations, some attempt should have been made to stiffen both these buildings in a longitudinal direction as this would have had a strong tendency to resist torsional racking of the buildings.

The Daily News Building has a tower 40 ft. square and about 255 ft. high. It was designed to resist a wind pressure of 20 lb. per sq. ft. in either direction at a unit stress of 24 000 lb. per sq. in. in the steel work. In the design of the wind-bracing, care was used in proportioning the details as well as the main members. There was a special effort to make the connections stiff as well as strong. Besides this, heavy interlocking tiles were used for the walls instead of partition tiles. This building suffered no structural damage. Based on these facts, the building should not show signs of failure before the elastic limit of the steel was reached, which is at about 36 000 lb. per sq. in. This is equivalent to a wind pressure of 30 lb. per sq. ft. over the entire surface without any assistance from walls.

The main lessons to be learned from the foregoing considerations are as follows:

1.—Adequate wind bracing is necessary in the construction of a building.

2.—The details must be carefully designed for the same strength as that for which the main members are calculated.

3.—In designing different types of wind braces for one floor level the designer should take the relative stiffness of the members into account and not depend too much on the arithmetical summation of the total strength of the connections as some members may be greatly overstressed before the others take any large proportion of the stress. On buildings of moderate size, however, it is permissible to take care of this point by the use of an adequate unit wind pressure.

4.—Stiffness as well as strength is a prime requisite for a good steel design. This is especially realized by the best engineers in designing narrow and high buildings used for living quarters, such as hotels and apartment houses, where the confidence of tenants in the stability of the structure is of the utmost importance.

5.—The effects of the Florida hurricane indicated that the floor system acted as a stiff plate or horizontal girder and that all columns were subject to the same horizontal deflection, except that the entire structure twisted, where one end of the building was stiffer than the other. In designing wind-bracing this must be given full weight and the bracing arranged, if possible, so that no twisting action from the wind can come into play. This means that the wind pressure at one end of a building and the bracing designed to resist it, must balance the relation between the corresponding pressure and bracing at the other end, so that the structure will deflect about the same amount at both ends. This principle applies especially where one end is higher or narrower than the other.

The following is quoted from a preliminary report made to the Structural Division on January 20, 1927, by the Chairman of this Committee:

"Summarizing the fields that this Committee might well cover are the following:

"1.—A determination of the wind pressure exerted by the storm.

"2.—A standard method for figuring wind stresses in tier building construction.

"3.—Standard details best able to resist wind stresses both from the standpoint of stiffness as well as that of strength.

"4.—A study to determine whether the same requirements for wind should be used for buildings having a low ratio of height to width as for buildings having a high ratio.

"5.—The strength of masonry walls and the bearing they have on wind bracing.

"I feel that there is a good chance of our accomplishing something valuable in connection with a part of these at least though it may entail more work than the Committee as now constituted can find time to perform. I am sure, however, were we able to cover any one item thoroughly, our work will have been well worth while."

Taking up the items mentioned in the order given:

1.—A Determination of the Wind Pressure Exerted by the Storm.—The Committee feels that, unfortunately, a strict determination of the wind pressure cannot be made. This has been covered earlier in the report. The results showed, however, that all the tall buildings consistently designed for a wind load of 20 lb. per sq. ft. survived without serious damage.

2.—A Standard Method of Computing Wind Stresses in Tier Building Construction.—The results of the storm showed that for buildings of the size and shape found in Miami, the common theory, as given in Fleming's "Manual on Wind Bracing", will produce structures capable of standing up under the most severe conditions. The failures were caused by neglecting to apply the theory consistently. On the other hand, buildings so designed will not necessarily be sufficiently stiff to give the tenants confidence in their safety. Moreover, in the case of very high or narrow structures, the obvious errors involved render the results less secure than is desirable. From a standpoint of strict theoretical determination of wind stresses, an adaptation of the slope deflection method will probably give the best results.[2] Some of the members of the Committee, however, are not entirely convinced that this method can come into universal use on account of the work that it will entail.

A consulting engineer generally has a limited time in which to design a structure, and if a method could be developed which will approach in results the slope deflection method and still entail very much less work, it would be desirable. One of the members of the Committee has used a method apportioning the amount of wind shear taken by the various columns on the basis of the relative moments of inertia of the columns. This is based on the assumption that the length of the column is the same. If the lengths between wind brackets vary considerably, as in the case of deep girders, these shears will have to be corrected to take care of the difference in column lengths. A closer approximation would take into account the effect of variations in the stiffness of attached floor-beams, which would involve practically the method developed by Professor Morris. The Committee feels that some study should be given to this before suggesting a definite method for computing wind bracing.

3.—Standard Details Best Able to Resist Wind Stresses Both from a Standpoint of Stiffness as Well as That of Strength.—Such details have been pretty well standardized by a few of the leading consulting engineers in New York, N. Y., and these could be released at any time. It would seem advisable, however, to release such information at the same time that a method for computing wind-bracing is developed and, therefore, no action has been taken in regard to this subject.

4.—A Study to Determine Whether the Same Requirements for Wind Should Be Used for Buildings Having a Low Ratio of Height to Width as for Buildings Having a High Ratio.—This item is also related to the method of computing wind-bracing and can well be taken up at such time as Item 2 is considered.

5.—The Strength of Masonry Walls and the Bearing They Have on Wind Bracing.—To secure any really valuable information on this subject, it would probably be necessary to have numerous tests made, which would entail considerable expense.

In the preliminary report before quoted, it was stated that to make a complete report might entail more work than the Committee, as now constituted, can find time to perform. To make an investigation along this line requires a very large amount of work, and few men are able to devote sufficient time to get results which entirely satisfy them. One of the members of the Committee, Mr. Miller, feels that a great deal of information as to wind pressure could be secured by means of self-registering barometers.

When the American Insurance Union Building was built in Columbus, Ohio, due to the initiative of Professor Morris, numerous gauge points were established on the columns and targets were located in the elevator shafts by which it is possible to measure the lateral displacement of the building during a storm for a height of 34 stories. No doubt a great deal of valuable information could be obtained by careful readings made on this building. Generally, owners are afraid to have any such tests made as they think it will cause the public to doubt the safety of the buildings; but in this case as the matter was all arranged when the building was constructed and the public now understands that work such as this is for educational purposes, the ordinary objections do not prevail.

---

[2] This is covered by the paper entitled "The Design of Tall Building Frames to Resist Wind", by Albert W. Ross, Jr., Jun. Am. Soc. C. E., and Clyde T. Morris, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1395.

The American Institute of Steel Construction, for several years has endeavored to advance knowledge along engineering lines in the use of steel. This question has been discussed with some of the members of the Institute, and it is possible that if its finances permit, it could be persuaded to establish a fellowship in Ohio State University to study the entire problem of wind bracing. In this case the Institute would probably be glad to have the work done under the general supervision of a committee of the Structural Division of the Society, provided that this would be agreeable to the authorities of the Ohio State University. Such supervision, of course, would involve some expense to the Society. In all probability, the student appointed to this fellowship would work under the direct supervision of Professor Morris, of this Committee. If this should be done, many new facts might be discovered in regard to wind-bracing which are now unknown, and a good student devoting his entire time to the subject under the proper supervision, could carry out many investigations which would entail far more work than any individual member of the Structural Division could devote to the subject.

In conclusion, the Committee hopes that this incomplete report may be of some value to the Engineering Profession in the design of wind-bracing. It wishes, of course, to stress the point that no system of rules, however complete, can take the place of thorough technical knowledge and long experience and judgment in work of this character; and that the work of designing wind-bracing for tall narrow buildings should never be entrusted to novices.

H. G. Balcom, *Chairman*,
D. C. Coyle,
F. R. McMillan,
Lee H. Miller,
Clyde T. Morris,
Committee.

January 19, 1928.

DISCUSSION

F. O. Dufour,[3] M. Am. Soc. C. E.—The report of the Committee is good. It covers all the points in an orderly and concrete manner. The speaker made an inspection of the effects of wind on buildings at Miami shortly after the storm.

The storm was as violent as any that has been registered; and on account of its passing over a city with many modern structures the opportunity for constructive criticism of the effects on usual engineering construction in areas of high wind velocity was exceptional.

What the actual velocities and pressures were appears to be immaterial, but nevertheless interesting. The speaker checked Professor Morris as to pressure when computed from the bending of columns in the front bent of the Meyer-Kiser Building, basing the wind distribution on the usual assumptions. This pressure was 60 lb. per sq. ft. Whether or not this was the pressure or whether the effects were due to synchronous behavior or those of impact due to sudden failure of the curtain walls is beside the question. The fact that the greatest destruction of the columns of the Meyer-Kiser building came after the walls were out appears to indicate that the bending was not due to the sudden failure of the walls. If the columns of the Meyer-Kiser Building had been designed for a pressure of 60 lb., even with the details used, it is most probable that no material distortion would have occurred.

The fact that wind-bracing in the form of knee-braces was used below the fourth floor and that no column distortion occurred below that floor, appears to indicate that had wind-bracing of this type been used throughout no column distortion would have occurred.

That the floors and ceilings were not even superficially cracked and that the rear of the building "kicked out" in a direction opposite to the front indicates that, as horizontal girders, these floors distributed the wind to the vertical bents in accordance with the usual supposition. Had the vertical wind bents, however, been equally spaced and had the wind pressure been uniform, it is probable that less damage would have occurred. That the wind pressure should have been uniform was, of course, not to be expected; that the strength of wind bents should be proportioned to stand a maximum difference in pressure is evident.

The report most properly calls attention to the insufficiency of details. The speaker noted a few broken connections and clip angles, but not more than three. A few more were cracked; many were bent.

In the case of the broken clip angles, one showed neither reduction of area nor elongation. Evidently, this break was due to causes other than the storm. Most of the bent connection angles showed no scaling. This indicated that the material was not stretched beyond the elastic limit. The partial springing back of the building when the walls were removed also indicated this.[4]

---

[3] Cons. Engr., United Engrs. & Constructors, Inc., Philadelphia, Pa.

[4] *Engineering News-Record*, the late E. A. Stuhrman, M. Am. Soc. C. E., December 29, 1927.

The stiffness of structures in areas liable to high wind velocity is, according to the report, next to strength, the most important consideration. The Committee has covered this in an efficient manner. Adequate interior wind bents and some construction for interior partitions other than plain partition tiles, together with a type of gusset-plate or knee-brace construction for the spandrel bents, are certainly required, especially when the depths of the spandrel beams are small.

An extended experience in the design of structures requiring stiffness as well as strength prompts the speaker to suggest that a design based on a unit stress of 9 000 lb. per sq. in. in tension, properly decreased for compression and increased for combined loads, will (in cases when the ratio of width to height is one-tenth or less) provide the desired stiffness, due regard being given to details sufficient to develop full strength.

It is believed that buildings designed in accordance with any of the methods commonly used by the leading structural engineers—a wind pressure of 30 lb. per sq. ft. and a due regard to details and wind-bracing—will, even with the present prescribed unit stresses, withstand injurious distortion as far as the steel frame is concerned. Consideration must be given to the element of stiffness, however, to prevent injury to the walls or inconvenience to the occupants. The floors will take care of themselves.

T. L. CONDRON,[5] M. AM. SOC. C. E.—It is stated that if a wind load of 20 lb. produced a stress of 24 000 lb., a 30-lb. wind load would produce a 36 000-lb. stress. The speaker believes that the 24 000-lb. stress was probably due to combined forces and that the difference between the unit stress of 24 000 lb. and 36 000 lb. would be due entirely to the additional wind load.

E. W. STERN,[6] M. AM. SOC. C. E.—The report of the Committee is interesting; it contains valuable information and suggestions, and it is hoped that its recommendations may be carried out so as to establish a fellowship at one of the engineering colleges, for a study of the entire problem of wind-bracing.

In this connection the speaker suggests that a series of tests be made to determine wind pressures on large surfaces. Few such tests are available and most of these were made on comparatively small surfaces, many years ago. In the light of present-day methods of experimentation, it is reasonable to suppose that much more accurate results would be obtained.

The usual type of beam and column connection illustrated in some of the structural handbooks, namely, a seat angle under the beam and a clip angle on top, has again been proved utterly inadequate in the Florida hurricane. The speaker has seen a building designed with such details pulled about 1 ft. out of plumb during erection, due to the thrust of a boom derrick handling a small load. Of course, thin angles in wind-bracing should never be used. They bend and pull away from the rivets and allow the columns to get out of plumb. This same criticism applies to light columns in which the component members are not heavy enough to transmit the load of the wind connections without bending or twisting, as exemplified in the buildings of Miami. All this is a matter of common knowledge among those who have experience in the supervision of the erection of the structural framework of buildings. Wind-bracing details require very careful study and, of course, should be designed with proper care as to stiffness in all directions. The speaker's practice has been to brace every bent longitudinally as well as crosswise with connections having side angles on every beam, even if the height and shape of the building are such that wind-bracing may not be necessary. During the erection of the steel, a framework braced by such details, is rigid, is easily kept plumb, and is not readily distorted by the swinging of boom derricks.

---

[5] Cons. Engr. (Condron & Post), Chicago, Ill.

[6] Cons. Engr., New York, N. Y.

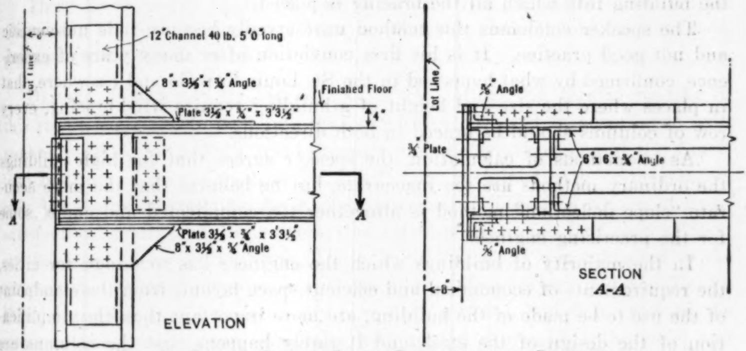


FIG. 6.—TYPICAL DOUBLE GIRDER CONNECTIONS TO COLUMNS.

The speaker offers for consideration the typical design shown in Fig. 6, which he often uses. It has proved to be very efficient in practice developing a large moment without projecting brackets, and the members are easy to erect.

The speaker was in the destructive tornado in St. Louis, Mo., in May, 1896, and has still a very lively recollection of what happened. There were 8 500 buildings damaged, 321 buildings totally wrecked, 255 lives lost, and it all happened within less than 15 min. Walls of the buildings which were wrecked usually fell outward; roofs were lifted and carried very great distances in some cases. The steel frame buildings—either finished or in course of construction—which were well designed, were not damaged, while many of the others with cast-iron columns and no wind-bracing were seriously impaired, some being completely destroyed. One such building was twisted about 1 ft., and another, a six-story building with cast-iron columns and brick walls laid in lime mortar, was pulled about 1½ ft. out of plumb.

The wind velocities were difficult to record because the weather gauges blew away, but it is the speaker's recollection that the highest velocity registered was about 120 miles per hour before the failure of the apparatus. A committee of very able engineers of St. Louis made an investigation immediately after the tornado and evaluated some of the wind pressures, among which was one derived from the study of a circular brick smokestack, 160 ft. high, which was blown over about 40 ft. above the ground. They found that this stack must have been subjected to a pressure of at least 85 lb. per sq. ft. static over an area 110 ft. high by 14 ft. in diameter. A very valuable paper[7] on the wind pressures in the St. Louis tornado was written by the late Julius Baier, M. Am. Soc. C. E.

There are apparently two different methods of designing wind-bracing that are in use among structural engineers. One of these, only a partial bracing of the bents, is exemplified in the buildings investigated in Miami. Quite a number of buildings in New York City have wind-bracing around the walls only, the theory being that if the floors act as horizontal girders they will transmit the wind pressure on the intermediate columns into the ends of the building into which all the bracing is placed.

The speaker condemns this method unreservedly because it is unscientific and not good practice. It is his firm conviction after many years of experience, confirmed by what happened in the St. Louis tornado and elsewhere, that in places where the size and height of a building require wind-bracing, every row of columns should be braced in both directions.

As to methods of calculation, the speaker agrees that for high buildings the ordinary methods are too inaccurate, but he believes that the more accurate "slope deflection" method is altogether too complicated and cumbersome for the practising engineer.

In the majority of buildings which the engineer has to design for cities, the requirements of economical and efficient space layout, from the standpoint of the use to be made of the building, are more important than the simplification of the design of the steel, and it rarely happens that the columns are spaced uniformly or even symmetrically about an axis. In such irregular cases the problem of calculating the stresses in the wind-bracing by the "slope deflection" method, or by any other accurate mathematical analysis, because of the many unknown quantities would involve so many simultaneous equations as to make the solution of the problem impractical, if not impossible. The case of the 48-story American Insurance Union Building, at Columbus, Ohio[8] is of interest in this connection. This building has six columns crosswise and was almost symmetrical about the axis, and was so assumed for the calculations. Other assumptions were made so that it was possible to get results which were correct for that part of the structure below the fifth floor only. This required, however, the solution of thirty-two simultaneous equations containing thirty-two unknowns. The time and expense involved in making these calculations would be prohibitive to an engineer in active practice even if he were an expert mathematician; and, furthermore, if the structure were not symmetrical about the axis a mathematical solution would be impossible.

For comparatively low buildings with several rows of columns the "cantilever method" is well worth considering. It seems to give safe structures and is quite simple to apply; but for high narrow buildings it is not sufficiently accurate, and it is necessary, therefore, to find some other way which will prove sufficiently accurate, and at the same time be much easier, more practical, and far less expensive than any mathematical method thus far developed. George E. Beggs, M. Am. Soc. C. E., has developed such a method, using models of cardboard or celluloid, and an interesting modification of this has been offered by Mr. Anders Bull, who uses brass rods in place of cardboard or celluloid models.[9]

---

[7] *Transactions*, Am. Soc. C. E., Vol. XXXVII (June, 1897), p. 221.

[8] *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1401, Fig. 3.

As regards the amount of wind load to use, the speaker believes that for tall buildings it is probably more nearly in accordance with actual conditions to use a higher wind pressure at the top than at the bottom, commencing, for example, with 40 lb. per sq. ft. for a 60-story building and reducing this to 10 lb. per sq. ft. at the level of the fourth floor.

These loads might apply to almost all districts throughout the United States, but in the hurricane or tornado districts, they should be increased, perhaps by one-third.

David A. Molitor,[10] M. Am. Soc. C. E. (by letter).—A careful reading of this report leads one to conclude that structures designed in accordance with modern practice for a 20-lb. wind pressure and the specifications of the American Institute of Steel Construction, would have sustained little or no irreparable damage by storms like the Florida hurricane. This is a source of great satisfaction to structural engineers, due fortunately to the fact that at least one fairly well-designed building (Daily News Building) was subjected to the severe test. Such damage as was done to the Meyer-Kiser and the Realty Board Buildings would undoubtedly have been avoided, had they received the same careful consideration, especially with regard to detailing the beam and column connections, as was given to the Daily News Building.

Therefore, the failures recorded cannot be attributed to lack of engineering knowledge respecting methods of wind analysis and designing of connection details, but must be charged to carelessness or ignorance on the part of designers who did not fully consider the information available to them. The writer has treated this subject more fully in a discussion[11] of another paper.

The following comments are offered relative to the five points suggested by the Committee:

1.—A Determination of the Wind Pressure Exerted by the Storm.—While this might be impracticable, the actual experience gained from designs based on a 20-lb. wind pressure seems to justify adhering to this practice; but why not use a 25-lb. pressure for this particular storm-infested area in future designs? Each locality has a Weather Bureau record of wind velocity covering a period of 10 to 20 years, from which the maximum can be selected for use, as in designing tall chimneys. From this maximum velocity, the pressure, in pounds per square foot, can be computed from the formula,

$$p = 0.0033\, V^2 \qquad (1)$$

in which, V is wind velocity, in miles per hour.

---

[9] *Engineering News-Record*, Vol. 99, December 28, 1897.

[10] Detroit, Mich.

[11] *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 189.

Records show maximum wind velocities ranging from 50 to 120 miles per hour, for various stations in the United States, but it would not seem prudent to use anything less than 60 miles per hour as a basis for computation even if weather reports gave a less value as the maximum. Except in some of the storm-swept areas on the Atlantic Coast and the Gulf of Mexico, 100 miles per hour is considered very conservative, and 87 miles is generally maximum for all inland points.

2.—A Standard Method of Computing Wind Stresses in Tier Building Construction.—Such a method has been used by many structural engineers, and it is difficult to understand why less rational methods should be used at all, or why more complicated methods, offering no greater reliability, should be proposed now.

3.—Standard Details Best Able to Resist Wind Stresses, Considering Both Stiffness and Strength.—It is in this connection that the most flagrant violations of good practice have been committed, of which examples described in the report are representative. There is no valid reason for this, because insufficiency in design means incompetency on the part of designers. Many important steel frames of to-day represent first attempts by amateurs.

4.—A Study to Determine Whether the Requirements for Wind Should not Be Made to Depend on the Ratio of Height to Width of a Building.—When strength alone is considered, the specified unit stresses govern every design. As rigidity is a factor of prime importance, however, and may vary between wide limits, it follows that the load assumptions and allowable unit stresses should bear some relation to the ratio of height to width of the frame. In speaking of rigidity of the steel frame alone, as affected by its width (neglecting the effects of the architectural fill materials), it should be noted that width governs the cantilever stiffness only when complete cross-frames or diagonal systems are used. For the so-called rigid frame, the basal width does not materially affect the cantilever stiffness which latter depends largely on the number of columns and the column details in the wind-resisting bent.

5.—The Strength of Masonry Walls and the Bearing They Have on Wind Bracing.—This the writer interprets to mean the effect of the architectural fill materials, including concrete, stone, brick, or tile, etc., composing the spandrel filling, floors, partitions, and fireproofing of the entire building. The inherent rigidity of the steel frame alone, as designed with specifications of the American Institute of Steel Construction, would not afford tenantable tall buildings when subjected to ordinary 60-mile wind. Deflection calculations prove this. However, experience teaches that this practice does produce commercially satisfactory buildings, and, hence, the architectural fill materials must be credited with supplying the extra rigidity required in such structures.

Perhaps in time some measurements of lateral building displacements may become available for the American Insurance Union Building, at Columbus, Ohio, in which instruments for taking such observations were installed during construction. This would be most gratifying, as no reliable measurements of this kind appear to be recorded.

The idea of instituting a fellowship to study the entire problem of wind-bracing, by appointing a good student to work under the direct supervision of

Professor Morris, is rather unique, especially in view of the closing remarks "that the work of designing wind-bracing for tall narrow buildings should never be entrusted to novices." It might have been more fitting to suggest the appointment of a committee of experts, trained by education and experience, to develop a practical and rational method of designing wind-bracing, so that a good student who is most likely a novice, could produce, with some practice, a creditable design.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1788

ICE AS AFFECTING POWER PLANTS[1]

FINAL REPORT OF COMMITTEE OF POWER DIVISION

WITH DISCUSSION BY C. M. DAILY, M. AM. SOC. C. E.

At the Annual Convention of the Society held in Chicago, Ill., in July, 1923, a paper was presented to the Power Division by William T. Walker, M. Am. Soc. C. E., on the subject "Ice Problems of Hydro-Electric Plants." In this paper Mr. Walker pointed out the need of the study of ice problems and urged the formation of a committee. At the conclusion of the discussion it was voted to appoint such a committee of the Power Division to study the subject of "Ice as Affecting Power Plants." The Committee appointed by Charles F. Loweth, Past-President, Am. Soc. C. E., (then President, Am. Soc. C. E.), consisted of the following members: Francis C. Shenehon, Minneapolis, Minn., *Chairman,* John R. Freeman, Providence, R. I., John L. Harper, Niagara Falls, N. Y., Daniel W. Mead, Madison, Wis., Louis C. Sabin, Cleveland, Ohio, and William T. Walker, Minneapolis, Minn.

The death of Mr. Harper in November, 1924, reduced the Committee to five. The geographical spread of the membership made meetings of the Committee difficult so that interchange of views has been by letter. The conception of the Committee of its best usefulness involved three elements: Observation, program-making, and compilation of facts as to existing practice. Consideration was given to the desirability of doing some research work, but none was undertaken.

At the Fall Meeting of the Society at Detroit, Mich., in October, 1924, a session of the Power Division was devoted to "Ice as Affecting Power Plants." Mr. Shenehon presented a brief report on the progress of work by the Ice Committee. An interesting talk was given by John Murphy, Electrical Engineer, Canadian Department of Railways and Canals, of Ottawa, Ont., Canada. Mr. Murphy's talk was illustrated by motion pictures and slides showing frazil formation and devices used to combat ice in hydraulic plants. A comprehensive discussion entitled "Ice Problems at Hydro-Electric Plants" was presented by Thomas H. Hogg, M. Am. Soc. C. E., Hydraulic Engineer of the Ontario Hydro-Electric Power Commission.[2] No activity of the Committee appears in the records of 1925.

---

[1] Presented at the meeting of the Power Division, New York, N Y., January 17, 1929.

In 1926 a search was authorized in the Engineering Societies Library under the following specifications:

(*A*) Ice jams in a river approaching a power plant, or jams in a river below, causing back-water rise and loss of head.
(*B*) Floating ice drawn into intakes or approach canals.
(*C*) Sheet ice forming in approach rivers, intakes, canals or forebays, including ice thrust on racks, gates, dams, and control works.
(*D*) Frazil as affecting intakes, control works, racks, and turbines.
(*E*) Snow, slush, or shell ice in ditches, flumes (as in the Sierras), intakes, control works, racks, and turbines.
(*F*) Combinations of Specifications (*A*), (*B*), (*C*), (*D*), and (*E*), in affecting power plants, forming on gates, racks, or dams.

Engineering treatments that have been devised to meet these troubles may be subdivided as follows:

Interception of sheet ice forming and breaking up in a head reservoir or lake (Specifications (*A*), (*B*), and (*C*)). This may be accomplished by the creation of slow current with consequent sheet-ice formation which serves as a barrier; or by intercepting works—as at Massena, N. Y.—formed of cribs and booms. It may also be accomplished by design of entrance of intake, canal, or forebay to by-pass the floating ice. The extent to which heat treatment may apply depends upon many conditions.

The physics of frazil (Specification (*D*)), including water temperatures, etc. (see works of Professor Howard T. Barnes, of McGill University, Montreal, Que., Canada); methods of combatting frazil; heat treatment; and forecasts of approaching trouble through surface temperatures.

Specifications (*E*) and (*F*) involve answers to the following questions: How far has the introduction of higher temperature water into forebays or collecting ponds lessened troubles in flumes or ditches? Will the housing of racks, with a temperature of 70° in the rack room, prevent the clogging of racks with frazil?

The search was completed in February, 1927. (See Appendix.)

In December, 1926, assignments were made to each member of the Committee on Ice as follows:

1.—Mr. John R. Freeman: The use of artificially produced heat, such as steam or warm air, to solve certain ice troubles in power plants.
2.—Mr. Daniel W. Mead: The solution of the problem of racks and their design to avoid ice troubles or to meet them.
3.—Mr. Louis C. Sabin: The ice conditions and troubles of the St. Marys River power plants and devices used in solving these troubles.
4.—Mr. William T. Walker: The ice troubles in the condensing water inlets of big steam plants and their solution.
5.—Mr. Francis C. Shenehon: The use of artificially produced heat, such as steam or hot air, to solve certain ice troubles in power plants.

In August and September, 1927, Mr. Shenehon visited hydro-electric installations in Norway and Sweden. In Sweden he visited the Porjus Development in Lapland north of the Arctic Circle and observed expedients used by the hydraulic engineers of these countries to meet winter conditions.

---

[2] *Canadian Engineer*, November 11, 1924.

At the meeting of the Power Division in New York, N. Y., on January 17, 1929, Professor Barnes presented a paper on "The Constitution of Water and Nature of Ice,"[3] and Mr. Walker presented a paper on "Ice in the Intakes of Steam Power Plants."[3] The Committee on Ice as Affecting Power Plants, following this meeting, submitted its final report and requested dismissal.

Perhaps in the logical sequence of a treatise on ice the fundamental fact is the nature of water and its transformations under heat variation. Professor Barnes has treated this subject in his paper previously mentioned. He has pointed out[4] that,

> "* * * water is an associated fluid, that its molecules are not simple H_2O molecules but molecular aggregates of complex form. Steam or water vapor consists of simple molecules which are called Hydrol. Water appears to be composed of double molecules called Dihydrol, while ice of triple molecules is called Trihydrol. From studies of the physical properties of water such as specific heat, density, and viscosity, it has been concluded that the associated water molecules of dihydrol contain varying proportion of the triple molecules trihydrol, and that the amount of the latter increases as the temperature falls from the boiling point. At the freezing point the proportion of trihydrol is estimated to be as high as 37% and at 100° C. about 16%.
>
> "Roentgen as early as 1892 stated that water contained ice in solution above the freezing point, and more recent studies have shown the identity of these ice molecules with the trihydrol. Before congealing, the water contains ice as a solution carries a salt, and the freezing point is the temperature where the solution becomes saturated with the ice and ready to produce the solid when the necessary energy is abstracted to enable the solid to precipitate out. To get this energy to flow out of the solution a small temperature depression is necessary, which we have already described. That the supercooling is not more pronounced is due to the rapid rate at which ice is formed tending to keep up the temperature so long as any water remains to form ice. Water supercooled by laboratory experiments to many degrees below 0° C. is a highly supersaturated solution of ice, and readily precipitates on being agitated or by introducing a small particle of ice. It is exceedingly interesting to observe the nature of this first precipitation. The particles of ice as soon as they can be seen under a microscope are devoid of crystal form and appear as a true colloid in small disc like particles. These flocculate and grow, passing through a crystal colloidal form to a true ice crystal. On allowing to grow slowly in quiet water they assume the form of a branching feathery snow crystal of most beautiful form. In rapidly moving water the crystals never become large, but are broken up and assume more and more a mass of broken spicules or spines. In Nature, we find this first precipitation of ice in water appearing as a cloudy formation throughout the mass of the water. This is the beginning of the fine ice noticed in rapids or open water and called frazil or needle ice. It is the form which gives the most trouble in power house operation, since it is produced all through the mass of the water and is of little or no buoyancy. It is readily carried into the racks and wheels, and adheres to the surface of machinery already cooled below the freezing point by the contact of the chilled water flowing through."

The illuminating statement that the flowing fluid water of a stream as it approaches the freezing point may contain more than one-third of its volume in the trihydrol or incipient ice form prepares the mind for a clearer understanding of the amazing phenomenon of the appearance of swarms of frazil, springing into existence when the agitated water drops less than 0.01° cent. below the freezing point. "Before congealing, water contains ice as solution carries salt!" Liquid ice is, therefore, a proper physical conception. The emergence of visible cloudy colloidal ice in water is likened by Professor Barnes to the visibility of mist emerging in an aqueous vapor. The delicate balance in the temperature of water passing from the liquid form to the colloidal and particle form lies safely within 0.01° cent. on either side of the zero line. If water is held to a temperature of 0.01° cent. no ice will form. Ice will form rapidly with a water temperature of — 0.01° cent. and agitation.

---

[3] Abstracts of these papers are published in *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, pp. 685 and 689, respectively.

[4] "Ice Engineering," Renouf Pub. Co., Montreal. First Edition, April, 1928, pp. 6–7.

The physical constants of ice are essential in this discussion. The density of clear ice may be taken as 0.92 and its weight at 0° cent. as 57.4 lb. per cu. ft. Where air is entrained in ice it is of course lighter. Water is known to increase in density as it is cooled until its maximum is reached at 5° cent. It then expands until its density at the freezing point is the same as its density at 8° cent.

The latent heat of the fusion for ice is 143.5 B. t. u. This means that the transformation of a solid pound of ice takes 143.5 times the heat necessary to raise the temperature of liquid water itself 1° Fahr.; and with 60% efficiency and coal having a heat value of 14 000 B. t. u., 1 lb. of coal will melt 59 lb. of ice; and 1 lb. of petroleum will melt 98 lb. of ice. The heat necessary to raise 1 lb. of water 1° Fahr. is the British thermal unit. This means that with coal of 14 000 B. t. u., 1 lb. of coal will raise the temperature of 14 000 lb. of water 1° Fahr. As a temperature rise of 0.1° will avoid freezing, 1 lb. of coal, therefore, used with 60% efficiency, will treat 1 340 cu. ft. of water. If this water is flowing through a steel siphon at the rate of 40 cu. ft. per sec., 1 ton of coal will serve for tempering to this extent this volume of water for 18 hours. Stated in another way if the temperature of the ditch water approaching a siphon is 32°.0, the addition of 0.1 cu. ft. per sec. of water at 70°, will raise the total volume to 32°.1 Fahr. and will be a safeguard against freezing.

The mechanical equivalent of heat may be taken as 1 h.p.-hour = 2 546 B. t. u. If this be taken in the form of electric current transmitted to the point of use at 70% efficiency this becomes 1 h.p.-hour = 1 780 B. t. u. Therefore, 1 h.p. will serve to raise the temperature of 28.5 cu. ft. of water 1° in 1 hour. The use of mechanically produced heat or electric current must be restricted to the maintenance of temperature of perhaps 32°.1 Fahr. in the metal of racks, gates, or turbine wheels, in order to avoid the adhesion of frazil.

The latent heat of the vaporization of ice is the sum of the latent heats of the melting of ice and the vaporization of water. This is about 143.5 + 1 076.5 = 1 220 B. t. u. The bearing of this constant has to do with evaporation from ice.

The specific heat of water is unity at 17°.5 cent., or 63°.5 Fahr. At 0° cent. the specific heat is 1.0094 and at 10° cent. it is 1.0023. The British thermal unit, therefore, applies strictly to water when its specific heat is unity. About 1% more heat is needed to raise the temperature of water at 0° cent. than at 17°.5 cent.[5]

The thermal conductivity of snow and ice is relevant in a study of water conditions in power streams. The thermal conductivity of ice, snow, and water is expressed in calorie-gramme-second units. For comparison the thermal conductivities of familiar metals are as follows:

Materials.	Thermal conductivity, in calorie-gramme-seconds.	Materials.	Thermal conductivity, in calorie-gramme-seconds.
Ice	0.005	Iron	0.11
Snow	0.0003	Silver	1.00
Water	0.0013	Steel	0.06
Aluminum	0.48	Zinc	0.26
Copper	0.89		

The blanket value of a snow-and-ice cover in a forebay pond is obvious.

The coefficient of expansion of ice is given by Professor Barnes as about 0.000 089. In the American Civil Engineers Handbook, Fourth Edition, a value ascribed to Ganot of 0.000 052 is given.

Professor Barnes has discussed[6] other physical elements, such as plasticity, elasticity, viscosity, strength, color, refractive index, electrical properties, and bacteriology. It appears needless and undesirable to include in this report a discussion of the physics of ice. The Committee on Ice in this report has utilized this work of Dr. Barnes on "Ice Engineering" to the fullest extent.

SURFACE FORMATIONS

It is desirable to suggest definitions for the various forms in which ice is related to power plants.

Sheet Ice.—The formation covering the water surface. In its simplest form, it is transparent even when a foot or more thick.

Cake Ice.—Solid sheet ice broken up by wind action, or other forces.

Border Ice.—Sheet ice forming in marginal bays where the water is shallow and the current slow.

Shore Ice.—The formation, along the banks of a stream with brush, driftwood, or débris incorporated.

Windrow Ice.—A consolidated aggregation of cake ice with edges up.

Honeycomb Ice.—Sheet ice rotting in place and eventually breaking down into long crystal needles.

Slush Ice.—Unconsolidated snow floating on the water surface after a snowstorm; or water-penetrated snow, covering and depressing sheet ice by its weight.

Snow Ice.—Consolidated slush ice.

Shell Ice.—Thin sheet ice forming and breaking up in such streams as the Ohio River above Louisville, Ky.

Floe Ice.—Bergs or masses from spray or water action.

---

[5] "Ice Engineering," by Howard T. Barnes, 1928, pp. 38–40.

[6] *Loc. cit.*, pp. 40 *et seq.*

MIDDLE FORMATIONS

Frazil or Needle Ice.—The swarming mass of spicules forming throughout the full depth of open, flowing, agitated, supercooled water. It is the precipitation in needle form of the solid ice, out of the liquid trihydrol element in the water.

Hanging Ice.—An agglomeration of frazil, anchor ice, slush ice, shell ice, and cake ice, forming on the bottom of sheet ice at the foot of a stretch of open swift-flowing water.

BOTTOM FORMATIONS

Anchor Ice.—A formation crystallizing on submerged objects, such as boulders, perhaps growing by the agglomeration of frazil.

In considering the adverse effects of ice on power-plant operation, the discussion may be assembled under the following headings.

A.—Lessened stream flow from a lake or reservoir because of ice formations in the outflow channel.
B.—Ice accumulations in reservoirs.
C.—Ice accumulations in approach canals and forebays.
D.—Lessened head from back-water caused by ice formation in outflow channels down stream from a power plant.
E.—Difficulties of gate operation in sub-zero weather.
F.—Ice formations or jams on spillway dams tending to block them.
G.—Ice formations on reservoir structures with varying levels in the reservoir.
H.—Ice thrust on hydraulic structures.
I.—Ice clogging of racks.
J.—Ice clogging of valves and turbines.
K.—Ice on the intake screens of steam plants.

It should be clearly understood first that the successful operation of a water power plant is not a matter of latitude or of the intensity of cold. The great State-owned power plant at Porjus, Sweden, lies 60 miles north of the Arctic Circle. In the deep lakes or fjords forming the reservoir and forebay pond of this plant, ice forms 3 ft. thick.

The Swedish engineers have built, still farther to the north, a multiple-arch dam—one of the most delicate of hydraulic structures—to conserve and regulate the flow of the Lule River. This river has an ultimate regulated minimum flow of 7 000 sec-ft. The Porjus development utilizes a fall of 184 ft., and has a present installation of 56 500 kw. and an ultimate capacity of 113 000 kw. The current is used for the electrification of the railroads of that region, for the operation of the iron mines at Kiruna and Gallivare, and for chemical industries. The forebay reservoir is deep, with a thick ice-and-snow blanket that keeps the water warm. The water is conducted through a deep, wide, slow-flowing open-rock cut intake canal, then through 1 700 ft. of tunnel in rock to a roof-covered distribution reservoir; then in vertical shafts in the rock, to the turbines in a power house carved out of solid granitic rock. The discharge tunnel 4 200 ft. long in rock leads to the lower levels of the river. The water being carried through rock from intake to tail-water freezing is completely avoided, even in extremely cold weather.

The same situation exists in the great Rjukan development on the Mana River in the mountains of Norway. Here, the water is conducted from a deep reservoir through rock-hewn tunnels to the three power plants; but in a development at Svaelgfoss farther down on the same river, ice troubles are severe. It was said that 10% of the current generated in sub-zero weather was used to solve ice troubles. The fundamental fact in the expectation of ice trouble is conservation of the heat in the water as it exists in deep reservoirs. In such deep reservoirs the bottom water, even in mid-winter, is not far from the temperature of the maximum density of water, 39°.2 Fahr.; and from that temperature it varies to about 32° at the bottom of the ice cover. The snow-covered ice acts as an insulator against the escape of heat and the earth crust temperature radiates from the bottom. Doubtless some heat from the sun penetrates the snow-and-ice insulation, and the freezing of the water on the under side of the sheet ice releases some heat by the process of molecular change. In deep, slow-flowing water, the rotation brings up the bottom strata and the mean temperature of the mass may be as low as 35° Fahr.

The obstruction of the outflow from a reservoir by ice formation in the stream is illustrated by the St. Clair River at the foot of Lake Huron. The normal summer fall in the 40 miles from Port Huron, Mich., to Lake St. Clair is 5.5 ft. When the sheet-ice cover forms in January, this 5.5 ft. of potential energy must serve the friction of the ice perimeter, in addition to the bottom perimeter of the open season. The result is a slower current flowing with lessened cross-sectional area. The outflow volume is at times only one-half the normal summer outflow. This was analyzed by Mr. Sabin, in the report of the Chief of Engineers, U. S. Army, for 1902. In this report, also, the lessened outflow of the great power stream, the St. Lawrence River, is analyzed by Mr. Shenehon, and in the report for 1900 he analyzes the lessened outflow of the Niagara River. The full water-power development of the St. Lawrence River, converting it into a series of slack-water pools where now turbulent rapids exist, may enlarge the winter flow.

The ice formations in a great reservoir like Lake Erie, which pass into the Niagara River in a procession of cake ice and floe ice, cause considerable trouble in the power plants at Niagara Falls. This ice may run up to February 1, when the foot of Lake Erie freezes over and forms a barrier against the entrance to the river of the drift ice of the open middle part of the lake. After the break-up of the sheet-ice barrier—perhaps in late March—the ice may run until mid-May. This run is largely cake ice with some ice floes, which ground in 8 ft. of water. In a paper entitled, "The St. Lawrence Waterway to the Sea",[7] by Mr. Shenehon, a project of breakwaters is suggested to minimize troubles in the Niagara River from Lake Erie ice. At the head of the Gota River in Sweden "an ice boom at the outlet of Lake Vanern prevents snow and ice from entering the river during northerly wind, reducing ice trouble."[8]

The entrance of floating ice into approach canals and forebays leads to many complications and difficulties. Where an approach canal is long, with a definite open-season loss of head, this loss is accentuated by the formation of sheet ice; or by packing with cake ice moving less rapidly than the water; or by the jamming of the ice. The by-passing of river or reservoir ice at the entrance to the canal or forebay is the fundamental solution. The principles of by-passing ice have been discussed[9] by Benjamin F. Groat, M. Am. Soc. C. E. The practice he suggested and utilized for many years is the separation of the surface water for the transportation and by-passing of ice in the direction of the flow of the stream, while the power water is drawn from the middle or bottom strata, preferably at right angles to the stream direction. Guides or vanes in the bottom or gathering tubes may facilitate the cross flow of the lower strata. The power water may pass under a curtain-wall built parallel to the direction of the stream. The velocity of the directed power water must be low enough to avoid pulling the ice under by suction.

---

[7] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 497.

[8] "Utilization of Water Power and Inland Navigation," by A. Ekwall, World Power Conference, 1926, Section 27, p. 7.

The approach to the long penstocks of the Ontario Company at Niagara Falls, built early in the Twentieth Century, illustrates the principle. The surface water carrying the buoyant ice approaching from the head of the rapids is carried past the curtain-wall and discharges with a depth of 2 to 3 ft. over weirs on the gathering-basin walls. A curtain-wall parallel to the Niagara River current guards the entrance to the long Chippewa-Queenston Canal, and the principle of deep-water draft is used, with gathering tubes contemplated. The approach to the ¾-mile canal of the Niagara Falls Power Company, is guarded by booms and by ice formations worked out by the late John L. Harper, M. Am. Soc. C. E., to by-pass the surface ice over the American Fall. At Massena, N. Y., on the St. Lawrence, ice diversion follows the principle of sub-surface intake. At Marseilles, Ill., on the Illinois River, the curtain-wall is utilized less effectively. When ice is running it is necessary to keep a gang of men at work to prevent the cake ice from passing under the wall. At best, some buoyant ice will enter canals and some ice will form in canals, and sluicing is necessary at the power house. A power sluice was used by the Niagara Falls Company. The present device, however, is the endless chain elevator method of mechanically lifting the ice from the water and dropping it into a chute.

It is best, however, to avoid floating ice and frazil by drawing power water from a deep ice-and-snow-covered pool. Mr. Freeman has proposed a dam at the head of the rapids at Niagara Falls below the intakes of the upper three power plants to raise the water 10 ft. This would mean slower current and possible ice cover in the river between Lake Erie and the Falls. When a current exists approaching a power canal, the use of booms may aid in the formation of a surface sheet. This is the practice in the Gota River, Sweden, at the 115 000-kw. power plant, in Trollhättan.

The lessened head caused by a back-water rise is illustrated by the conditions in the Grass River down stream from the power plant at Massena. At the foot of the Lachine Rapids of the St. Lawrence River at Montreal, Que., Canada, a winter rise of 12 ft. is due to ice cover in the river down stream and the formation of huge hanging dams of frazil, anchor, and débris ice. The same cause sometimes obliterates the normal 11-ft. summer fall of Rapide Plat at Morrisburg, Ont., Canada, and Waddington, N. Y. The back-water here has been accentuated at times by the farmers along the river swinging a field of sheet ice, formed in a bay, across the current to form a bridge. In the Illinois River below the power plant at Marseilles, the formation of windrow ice and hanging dams at times obliterates the 11-ft. fall. The complete power development of a river by replacing rapids with slack-water deep pools will remedy the back-water. A deepening and straightening of the channel of the Grass River below Massena bettered the back-water conditions.

---

[9] "Ice Diversion, Hydraulic Models, and Hydraulic Similarity," *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1138.

The difficulties of gate operation under ice conditions are met by the use of live steam—which may be generated in a boiler electrically fired. The use of warm compressed air bubbling against the up-stream side of the gates helps in part and avoids ice thrust. One of the 65-ft. roller gates of the Solbergfoss Plant of the City of Oslo, Norway, is electrically heated for emergency winter operation. This type of gate is readily warmed by a contained heating unit.

The spillway dam appurtenant to a power plant in a stream of heavy ice runs, especially if it be also a flood stream, needs the longest uninterrupted crest length. The roller gate, 20 m. long, previously mentioned, has many advantages over the relatively short-span Taintor or Stoney gates. Better yet are wicket gates which leave when down a clear crest line. The avoidance of ice formation on the crest of the hollow concrete Norrfors Dam is accomplished by steam pipes passing through the chambers, from a boiler electrically fired. At the Hammerfossen Dam the piers, instead of having cut-water prows, have a flat up-stream end, the water cone formed serving effectively to avoid the lodging of logs or ice. These two dams are in Sweden. Robert Fletcher, M. Am. Soc. C. E., has painted[10] a vivid picture of the combination of ice and flood:

"At Sumner's Falls a rocky barrier extends so obliquely across the river that its length is nearly double the direct width of the river. About 50 years ago at this site there was a dam, a canal lock and approach canal for river boats, and a very large saw-mill running seven saws. A spring flood brought down ice which jammed and froze; a second flood increased the jam, piled the ice high on the dam and against the banks, and finally carried the mill down stream, and wrecked the dam and lock, which were never rebuilt. The writer has seen an impressive picture of blocks of ice up to 4 ft. thick (a man standing beside one) wedged together over an extent of many acres, on one side of the Lower Yellowstone Dam, in Montana, as the result of a high flood. This suggests in part the possibilities of destruction by a spring flood carrying ice."

Ice formation in mountain reservoirs in February with continuous pull-down involves the spalling and disintegration under freezing of concrete dams. In a paper[11] by Fred O. Dolson and Walter L. Huber, Members, Am. Soc. C. E., remedies for frost action are stated. For a multiple-arch dam it would appear simple and not too expensive to carry down an insulating wall or diaphragm on the down-stream side, so that the bays are enclosed.

---

[10] *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), pp. 1161–1162.

[11] "Multiple-Arch Dam at Gem Lake on Rush Creek, California," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 713.

Maintaining, by salamanders or otherwise, a temperature of 33° Fahr. within the chamber should safeguard the up-stream face against ice adhesion. Heat treatment in such a case will be more effective than super-waterproofing. The heating of the hollow Ambursen type dam at Norrfors, Sweden, has already been mentioned.

Ice thrust on the up-stream faces of dams, unless intercepted, involves great forces. Professor Barnes[12] has given the following values:

Ice thickness, in inches.	Thrust, in pounds per linear foot.
6	28 800
8	38 400
10	48 000
12	57 600

These values are based on a crushing strength of 400 lb. per sq. in. Quoting from Mr. George G. Bell, the ice thrust is given for 18 in. as 43 200 lb. per lin. ft. Mr. Bell concludes that a dam exposed to full ice pressure should be designed to resist from 43 000 to 48 000 lb. per lin. ft.[13]

A paragraph on "Ice Thrust" in the American Civil Engineers Handbook[14] is as follows:

> "Ice pressure acting at the top of the dam is sometimes required to be taken into account. John D. Van Buren in 1895 advocated the use of 40 000 lb. per lin. ft. The board of experts for the Quaker Bridge dam (report of Aqueduct Com., 1889) recommended that the ice pressure should be taken at 43 000 lb. per lin. ft. of dam. For the design of the Kensico Dam 47 000 lb., for the Cross River dam 24 000 lb., and for the Croton Falls dam 30 000 lb. per lin. ft. of dam have been used. These widely differing specified values of the ice pressure are mostly based on the crushing strength of the ice, but it is obvious that before pressures of such magnitude could be developed, the ice sheet would fail as a long column; moreover the creeping of the ice up the generally sloping shores of the reservoir will prevent any considerable thrust upon the dam. It can be shown (Trans. Am. Soc. Civ. Eng., Vol. 75, p. 164) that the consideration of ice pressure of such intensities as quoted above would result in absurdly large dimensions for low dams while the dimensions of high dams would be only slightly affected. It is believed unnecessary to consider ice pressure in the design of dams unless the reservoir is very short in the direction perpendicular to the axis of the dam and the sides slope abruptly."

The ice thrust assumed in the design of the Lule River (multiple-arch) Dam in Sweden above the Arctic Circle, where the temperature falls to 50° below zero Fahrenheit and ice becomes more than 3 ft. thick, was less than 40 000 lb. per lin. ft. It is desirable to maintain by hand-sawing a gap in the ice sheet parallel to the dam, or to utilize bubbling compressed air. Even where wicket dams are used to keep up the winter head an open slot in the ice sheet avoids thrust.

Where metal racks reaching above the water surface are exposed to subzero weather in frazil-forming streams, the probability of complete blocking is great. Wherever this probability exists the racks themselves, or their supporting members, or both, should be designed to support the full load of the static head. Where the racks are enclosed in a heated rack room or rack chamber, it is still desirable to design the supports to carry the full static head, since a collapse means a long shut-down for repairs. The heating of the racks in some form so that ice will not fasten to them is desirable. The simplest form is the heated rack chamber made by decking over the rack space. Heat may be supplied best (as it is done in the Norrfors Plant in Sweden) by providing a conduit that carries the hot air from within the generator casing to the rack chamber. A curtain-wall is desirable to by-pass floating ice. In some rack rooms where the air temperature is but little above 32° Fahr., no trouble has resulted. However, in rivers, such as the Gota River in Sweden, where frazil forms and agglomerates and comes to the racks as huge slushy masses it is desirable to utilize additional safeguards. The fine screens are removed during frazil periods. The remaining coarse racks have clear openings between bars of 6 in. or more, and, in addition, electrical heating is ready for emergency use.

---

[12] "Ice Engineering," p. 214.

[13] *Proceedings*, Maine Soc. of Civ. Engrs., Vol. I, 1911, p. 41.

[14] Fourth Edition, p. 709.

The Chief Engineer of Water Power in Sweden, Mr. A. Ekwall, estimates the electric current consumed in heating the racks as that coming from a 2-m. fall of water. Where the operating head on a plant is 20 ft., the energy consumed in heating the racks is 30% of the output; while with a head of 60 ft., it is only 10 per cent. The greatest safety against ice troubles lies in enclosed heated rack chambers, wide spacing in racks, and provision for emergency heating of racks. It should be noted that the use of electrical current is occasional and is likely to be at off-peak times. Should the racks be placed so that they may be flooded with sunshine, the condition is better still. The avoidance of night radiation and the absorption of heat by daylight are the ideal conditions. The complication of anchor ice rising in masses from the bottom, carrying sand, gravel, and boulders, and moving below the surface, complicates by-passing, reliance on curtain-walls, and the removal of the lower level racks.

The clogging of valves and turbine casings by massed frazil, slush ice, or anchor ice, is perhaps present in low-head plants with small units on horizontal axes. Where such clogging occurs, the use of live steam is effective in restoring operation. Mr. Hogg suggests concrete turbine casings "with series immersion electric heaters."[15] The use of electric current in the form of the water rheostat has been effective to break up ice masses. Bubbling warm air is also used. Professor Barnes advocates the use of thermit.[16]

It is understood that compounds more effective than thermit are in course of development. Other forms of chemicals are used, such as calcium chloride. Flooding with heat rays is also in use. It is perhaps desirable to accentuate the higher effectiveness of preventive measures to avoid clogging, rather than the surgical methods of breaking up the ice after a shut-down. So a little heat is needed—perhaps, a few thousandths of a degree—to avoid ice adhesion to metal structures, that the best practice is mild heating of dangerous parts.

Ice troubles in the screens of condensing-water intakes in steam plants and the remedies have been discussed by Mr. Walker[17] who points out the superior

---

[15] *The Canadian Engineer*, November 11, 1924, p. 497.

[16] In *Engineering Journal*, The Eng. Inst. of Canada, November, 1926, Professor Barnes discusses the nature of thermit ice reaction.

[17] Abstract, *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, p. 689.

economy of re-circulating the condensing water. The breaking of ice jams in power streams is an art in itself, involving the use of dynamite, thermit, salt, ash, sand, and gravel.[18]

In concluding this report the Committee may summarize by saying that ice troubles are consolidated cold troubles; that heat is the instrument to prevent trouble; that heat is present in the water itself conserved in deep ice-and-snow covered reservoirs; that daylight and sunshine are great heat resources; that the hot air inside the generator casings of power plants is a by-product to be utilized perhaps more effectively than heretofore; that discharged condensing water in steam plants may in part be re-circulated to temper the intake water; and that electric energy itself, in its thermal capacity, is a facile instrumentality—and not too wasteful at off-peak hours of the night or at daybreak.

FRANCIS C. SHENEHON, *Chairman,*
JOHN R. FREEMAN,
DANIEL W. MEAD,
LOUIS C. SABIN,
WILLIAM T. WALKER,

January, 1929. *Committee.*

APPENDIX

BIBLIOGRAPHY OF ICE IN HYDRO-ELECTRIC PLANTS

1881—Francis, J. B. Anchor Ice. 1881. (In *Transactions*, Am. Soc. C. E., Vol. X (1881), pp. 192-194; part of Presidential Address).

Discusses the nature of anchor ice and its interference in the operation of water power plants.

1882—Report on Ice Harbor at the Head of Delaware Bay. 1882. (In Annual Rept., Chf. of Engrs., U. S. A., 1882, Pt. 1, pp. 785-790.)

Several paragraphs on ice pressure, followed by tables on crushing strength of ice, from experiments at Frankford Arsenal, Philadelphia, Pa., and at Fort Tompkins, Staten Island, N. Y.

1884—Ludlow, William. Observations on the Crushing Strength of Ice. 1884. (In *Proceedings*, Engrs. Club of Philadelphia, v. 4, pp. 93-99.)

To determine the resistance necessary to be provided against ice in constructing piers for an ice harbor at the head of Delaware Bay.

1891—Dumble, J. H. Some Observations on the Expansion and Contraction of Ice on Canadian Waters. 1891. (In *Transactions*, Canadian Soc. C. E., v. 5, pp. 270-308.)

Experience of the effect of ice movement on bridge piers.

——Abstract. Movement of Bridge Piers by Expanding and Contracting Ice. 1893. (In *Engineering News*, v. 29, pp. 41-42.)

——Editorial. 1893. (In *Engineering News*, v. 29, p. 37.)

1893—Failure of Small Dam. 1893. (In *Engineering Record*, v. 27, p. 400.)

Failure of dam by ice belonging to Des Moines Water Power Co., during low-water period. Dimensions of dam not given.

1894—Freeman, John R. Hoisting Apparatus of the Canal Head-Gates at Sewall's Falls, N. H., 1894. (In *Transactions*, Am. Soc. C. E., Vol. XXXII (1894), pp. 278–294; Discussion, pp. 294-332.)

Pages 295, 304, 306-320 contain data from actual experience on the formation of anchor ice on canal head-gates.

1897—Hudson, H. B. Effects of Mississippi River Floods on New St. Anthony Falls Dam. 1897. (In *Engineering News*, v. 37, p. 290.)

Failure of St. Anthony Dam, due to ice. Gorge formed 20 miles up stream, broke. Dam approximately 250 ft. long, of which 110 ft. were swept away by ice pressure. Photographs of the dam, before and after failure.

---

[18] *Engineering Journal*, The Eng. Inst. of Canada, November, 1926, contains descriptions by Professor Barnes of the "Breaking of the Allegheny Ice Gorge of March, 1926."

1898—Leonard, R. W. Masonry Pier Moved by Ice and Replaced. 1898. (In *Transactions*, Canadian Soc. C. E., v. 12, pp. 131-134.)

The thrust of the ice was about 11 tons per sq. ft., considering the ice 3 ft. thick and striking the pier obliquely on a width of 5 ft.

1899—Failure of a Minneapolis Dam by Ice Pressure. 1899. (In *Engineering Record*, v. 39, pp. 542-543.)

Plan of Minneapolis Dam given in diagram; width of dam was 12 ft. at base and 5 ft. at top; 250 ft. long and 18 ft. high. Dam owned by St. Anthony Power Co.

——Rickey, W. Failure of Dam at Minneapolis Due to Previous Weakening Through Ice Pressure. 1899. (In *Engineering News*, v. 41, p. 307.)

Illustration showing trench cut in ice along face of the dam. Dimensions of dam, 535 ft. long, controlling falls of the Mississippi River, at Minneapolis. Sectional view of dam and mill of the Minneapolis Mill Co.

——Severe Test of Bridge Masonry. 1899. (In *Engineering Record*, v. 40, p. 554.)

Effect of a mass of ice sweeping against a bridge, carrying away a span and moving a pier off its foundations.

1903—Cairns, R. A. Wigwam Reservoir Masonry Dam of Water-Works of Waterbury, Conn. 1903. (In *Engineering News*, v. 49, pp. 418-419.)

General description of the dam, including dimensions, architectural design, with photograph and plan view. Conditions existing up stream and general description of surrounding territory, character of soil, water movements.

1905—Canadian Niagara Power To-Day. 1905. (In *Electrical World*, v. 45, pp. 17-20.)

Illustrates a set of iron ice racks, partly above and partly below the water level at the intake.

1906—Barnes, H. T. Ice Formations with Special Reference to Anchor Ice and Frazil. 1906. (Pub. by Wiley, N. Y., 260 pp.)

Discusses physical laws governing the transfer of heat, physical constants of ice, formation and structure of ice; the nature of sheet, frazil, and anchor ice; precise temperature measurements; theories to account for frazil and anchor ice and methods of overcoming the ice problem in engineering work.

——Patton, W. M. Practical Treatise on Foundations. 1906. Second Edition. (N. Y., Wiley, pp. 49, 107-113.)

Gives brief description of several cases showing effect of drift ice on the stability of piers.

1908—Dibos, M. Glaces de Mer, Glaces de Rivières, Manoeuvres de Preservation et de Deglacage. (Protection from breaking of sea and river ice.) 1908. (In *Memoires et Traveaux* de Société Ingénieurs Civils de France, 1908, Pt. 2, pp. 860-923; Abstract, *Minutes of Proceedings*, Inst. C. E., Vol. 179, pp. 441-442.)

Considerable space is devoted to descriptions of the various forms of ice and the phenomena of freezing. Illustrated descriptions are given of protective mechanism designed by an officer in the Russian Navy. The conclusions reached by the Congress of Navigation at St. Petersburg, regarding the best means of protecting structures from ice, are given. The oxy-hydrogen flame is suggested as a means of thawing grooves in solid river ice. Superficial and submerged charges of dynamite, melinite, black powder, or Favier safety powders may be used. Their advantages are discussed.

1911—Harrison, C. L. Provision for Uplift and Ice Pressure in Designing Masonry Dams. 1911. (In *Transactions*, Am. Soc. C. E., Vol. XXXVII (1897), p. 142.)

The question of uplift discussed with reference to types of construction for bridges and dams. Ice pressure on reservoirs and the necessity of considering this factor in connection with bridge or dam design.

1912—Failure of Dam from Ice Pressure. 1912. (In *Engineering News*, v. 65, p. 681.)

Failure of dam, 150 ft. long, 21 ft. high, and 15 ft. wide, belonging to Port Henry Light & Power Co., Rockport, N. Y. Ice, 3 ft. thick.

——Partial Failure of a Masonry Dam, Caused by Ice Pressure. 1912. (In *Engineering Record*, v. 65, p. 94.)

Partial failure of stone dam, on Saranac River, N. Y., due to ice pressure. Dam, 152 ft. long and 45 ft. high.

——Wall, E. E. Ice Trouble at the St. Louis Water-Works. 1912. (In *Engineering News*, v. 67, pp. 212-213.)

Discusses the use of a clamshell bucket to remove ice from the intake of the water-works plant.

1914—Barnes, H. T., Hayward, J. W., and McLeod, N. M. Expensive Force of Ice. 1914. (In *Proceedings*, Royal Soc. of Canada, Series 3, v. 8, Section 3, pp. 29–48.)

Table of coefficients of expansion of ice; observations made by the authors on the ice in the Montreal filtration chambers, 1914; conclusions as result of preliminary study of ice expansion, it is found that: (1) the crushing strength of ice is most probably 400 lb. per sq. in.; (2) an ice block will yield under pressure at approximately 200 lb. per sq. in., which is probably due to the slipping of the crystals; (3) an ice sheet will form cracks on the upper and under surface due to unequal strain; (4) that a permanent expansion may result if the cracks become filled and frozen; (5) according to the most trustworthy results of other observers, the ice frozen to concrete develops its full crushing strength, and the tensile strength of ice is less than 200 lb. per sq. in.

——Fuqua, P. D. Breaking a Heavy Ice Jam in the Great Miami River. 1914. (In *Engineering News*, v. 71, pp. 997-998.)

Describes the diversion of a serious ice jam from bridge and dam at Hamilton, Ohio, by setting off large explosive charges. Brief data are given on the damage done to structures by ice scour.

——Hendry, M. C. Ice Troubles in Hydro-Electric Plants. 1914. (In *Canadian Engineer,* v. 27, pp. 693-694.)

Discusses the nature of sheet, frazil, and anchor ice and the effect of each type on the winter operation of hydro-electric plants.

——Holtwood Dam Weathers Early Ice Run in Susquehanna River. 1914. (In *Engineering Record,* v. 69, p. 421.)

The Holtwood Dam is said to have had a deterrent effect on the ice jams in the river below.

——La Motte, A. Dynamiting Ice Jams. 1914. (In *Engineering Record,* v. 69, p. 365.)

An effective means of breaking river ice is described. Submerged charges are considered most effective.

——Winter Operation of Stoney Gates. 1914. (In *Engineering Record,* v. 70, p. 691.)

Describes the application of steam conduits to the large dam gates at the Shawinigan Water & Power Co.'s hydro-electric plants on the Maurice River. Diagrams show the extent of ice formation on piers and gates.

——Barnes, H. T. Crushing Strength of Ice. 1914. (In *Transactions,* Royal Soc. of Canada, Series 3, v. 8, Section 3, pp. 19-22.)

C. A. Mees concluded that the crushing strength of ice might reach as high as 800 lb. in a short column. He says, however, "Wherever the effects of ice expansion have been carefully observed, it has been noted that bending takes place, and this may be expected because we have a long column effect. On this account assuming one-half the crushing strength may be developed before bending, the maximum thrust may equal about 400 lb. per sq. in." The author states: "Probably the best experiments have been made by Mr. George G. Bell; if we neglect his highest result, which appears to be too high, we find values more consistent, ranging from 358 to 783 lb." Details of author's experiments are given.

1915—Allner, F. A. Ice Fighting Is Systematized at Holtwood Hydro-Electric Plant. 1915. (In *Engineering Record,* v. 72, pp. 66-68.)

Describes the precautions taken at the Susquehanna River plant to prevent shut-down of main and exciter units due to frazil and anchor ice. Steam is used to warm turbine chamber.

——Allner, F. A. Severe Frazil Ice Attack at Holtwood Plant Does not Disturb Service. 1915. (In *Engineering Record,* v. 72, pp. 113-115.)

Some detailed information on the first two onsets of frazil ice and the damage it incurred; at the time of the third occurrence the measures for protection had been put into practice. The situations are compared.

——Bligh, W. G. Dams and Weirs. 1915. (Am. Tech. Soc., Chic., 206 pp.)

Discussion concerning the necessity of considering ice pressure in designing dams, bridges, and reservoirs.

——Smith, C. W. Construction of Masonry Dams. (McGraw-Hill, N. Y., 272 pp.)

The subject of ice thrust is discussed on pp. 117-126, giving specific example of dams and reservoirs which have been successful and others which have failed.

——Use of Special Gates to Prevent Ice Formation at Intake Screens. 1915. (In *Electrical World,* v. 66, pp. 1030-1031.)

Illustrates hand-operated screens installed at the Dundee Falls, Me., plant of the Presumpscott Electric Co.

1916—Morrison, C. E., and Brodie, O. L. Masonry Dam Design. 1916. (John Wiley Sons, N. Y., 267 pp.)

Pages 16-19 discuss the question of ice thrust in reservoirs and dams using specific examples. Pages 115-119 work out, according to the formulas, the proper design for dams, allowing for ice pressure.

1917—Creager, W. P. Engineering for Masonry Dams. 1916. (John Wiley Sons, N. Y., 229 pp.)

Ice pressure is discussed, pp. 37-39, including the subject of ice thrust and allowances which must be made when designing a structure. A table gives ice pressures adopted for various dams; the design of reservoirs. Pages 78-104 include examples showing the necessary allowances to be made in building to account for ice pressure.

——Preventing Ice Troubles at a Connecticut Hydro-Electric Plant. 1917. (In *Electrical World,* v. 69, pp. 184-185.)

Illustrates the arrangements constructed for disposing of ice that clogs the forebay.

——Smith, J. C. The Hydro-Electric Power Plant of the Cedars Rapids Manufacturing & Power Co., at Cedars, P. Q. 1917. **(In *Canadian Engineer,* v. 32, pp. 211-213; Abstract from *Transactions,*** Canadian Soc. Civ. Engrs., article not available.)

A sketch is given to show the general plan and layout of the embankment. A description is given of the power house and the main units, as well as of the method used in diverting ice from the intake.

1918—Daily, C. M. Keeping Ice from Ports of Water Intake. 1918. **(In *Engineering News-Record,*** v. 80, pp. 900-901.)

Describes the plan adopted by the St. Louis Water-Works authorities to protect intake ports. The device illustrated consists of a timber boom with buoyancy cylinders.

——Davis, A. Protection of Keokuk Dam Gates from Ice Pressure by the Use of Air. 1918. (In Stone & Webster *Journal,* v. 22, pp. 199-206.)

A history of the ice action at Keokuk, together with some conjectures as to its causes, followed by a description of a very successful air system for dealing with ice blocking.

——Groat, B. F. Ice Diversion, Hydraulic Models, and Hydraulic Similarity. 1918. (In *Transactions,* Am. Soc. C. E., Vol. LXXXII (1918), pp. 1138-1151, **and discusssion.)**

An illustrated paper which treats of a new method of diverting surface water and all floating materials carried thereby, for the purpose of preventing jams in canals and rivers.

——Installing Apparatus to Protect Dam from Heavy Ice. 1918. (In *Electrical World,* v. 71, p. 778.)

Describes the use of a 5-h.p. air compressor to prevent ice formation on spillways and gates of dams and hydro-electric plants.

——**New Installation Keeps Ice Away from Keokuk Dam.** 1918. (In *Engineering News-Record,* v. 80, pp. 821-823.)
A pipe line across the Mississippi River supplies a constant current of air up stream and prevents the formation of ice on the gates.

——**Pierce, C. H.** Effects of Ice on River Discharge. 1918. (In *Journal,* Boston Soc. C. E., v. 5, pp. 414-419.)
Stream flow for the canal and weir types of waterways is computed, theoretically. Using these calculations as a basis the effects of frazil, surface, and anchor ice cover are worked out. Graphical results show the effect of ice on river discharge.

——**Purcell, J. M.** Ice in Hydro-Electric Plants. 1918. (In *Power,* v. 48, p. 951.)

——**Slush Ice Prevention at the Denver, Colorado, Water-Works Intake.** 1918. (In *Engineering News-Record,* v. 80, pp. 615-616.)
A sheet-piling channel in the forebay and the maintenance of a steady flow of water through a 60-in. conduit prevents the formation of ice around the intake screens.

1919—Bromley, C. H. Operation at Holtwood. 1919. (In *Power,* v. 49, pp. 850-854.)
Describes the manipulation of screens and trash racks to prevent ice clogging at turbine intake. The racks are made in sections, one or two sections being removed. This article discusses at some length the difficulties encountered with débris and how the condition was treated.

——**Russell, S. R.** Method of Breaking Up Ice Gorges with Dynamite. 1919. (In *Engineering and Contracting,* v. 51, pp. 217-218.)
Describes and illustrates three ways of placing dynamite charges for varying types and positions of ice jam.

——**Stickle, H. W.** Monongahela River Navigation. 1919. (In *Proceedings,* Engrs. Soc. W. Pa., 34, pp. 245-266; Discussion, pp. 267-277; Abstract, *Engineering and Contracting,* v. 50, pp. 283-284.)
Pages 252-253 describe methods used in dynamiting ice blocks in the vicinity of Lock No. 5.

——**Wilson, R. M.** Design of Hydro-Electric Plants for Combatting Ice Troubles. 1919. (In *Journal,* Eng. Inst. of Canada, v. 2, pp. 383-395; Discussion in *Canadian Engineer,* v. 36, pp. 237-242.)
An article showing how ice troubles at turbine intakes have been eliminated by the Cedars Rapids Manufacturing & Power Co. A special type of trash rack is described, which is built in sections, allowing a portion to be removed. A considerable amount of railroad ties, stumps, stones, and other débris was found in the runners, but this condition was satisfactorily met. Discussion follows the article.

1920—Balderston, C. Ice Inside Cast-Iron Column Raises Floor. 1920. (In *Engineering News-Record,* v. 84, p. 1166.)
Relates a case of ice formation inside a cast-iron column which resulted in lifting the column several inches off its base and pushing a heavy granite sidewalk a like distance.

——**Carpenter, J. S.** Ice Protection for Hydro-Electric Plant. 1920. (In *Power Plant Engineering,* v. 24, pp. 1125-1126.)
Explains methods for deflecting ice jams. Ice forms around speed rings of turbine, causing water passages to become clogged.

——**Concrete Road Uninjured by Weight of Ice Jam.** 1920. (In *Engineering News,* v. 85, pp. 29-31.)
Gives example of Clark's Ferry at junction of Juanita River with Susquehanna, where a concrete pavement remained undamaged after a heavy ice jam.

——**Eckhard, G. F.** Large Highway Bridge Wrecked by Pressure of Cake Ice. 1920. (In *Engineering News,* v. 84, pp. 902-904.)
Gives example of the Brattleboro (Vt.) steel span of 330 ft. pushed off its supports by floating ice. Weight of steel in bridge was 270 tons. Distortion of floor system increased until horizontal deflection of the lower lateral system became at least 2 ft.

——**Groat, B. F.** Ice Diversion for St. Lawrence River Power Co. 1920. (In *Canadian Engineer,* v. 39, pp. 545-552; Abstract in *Engineering and Contracting,* v. 55, pp. 45-46.)
Discusses construction of submerged weir and ice diversion works in South Sault Channel of St. Lawrence River. Ice jams cause high back-water, thus reducing the head on the turbines during such periods, sometimes by as much as 10 ft.

——**Merriman, M.** Ice Pressure. 1920. (In Am. Civil Engr's. Handbook, 4th Edition.)
Example given of a sheet of ice 150 ft. in width tipping a masonry bridge pier weighing 1 000 tons, 2 in. out of plumb (p. 435). Pressures due to ice have been found to be 30 000 lb. per sq. ft. for field of melting ice 1 ft. thick, striking a pier and crushing its way past (p. 709).

1921—Murphy, J. Ice Formation and Its Prevention. 1921. (In *Canadian Engineer,* v. 40, pp. 291-292, 294.)
A review of the progress of investigations into methods for the prevention of the formation of ice in hydro-electric practice. States that it is possible to prevent ice from adhering to valves, gates racks, penstocks, turbine runners, and casings by the proper use of heat.

1922—Gaby, F. A. Queenston-Chippewa Development of the Hydro-Electric Power Commission of Ontario. 1922. (In *Journal,* Am. Inst. Elec. Engrs., v. 41, pp. 508-526.)
Discusses the design of the intake, taking into account the floating and frazil ice which forms on the Niagara and Welland Rivers. A submerged intake consisting of gathering tubes and a surface intake forming a concrete barrier to ice, are part of the layout which is proving successful.

——**Ice Handling Methods.** 1922. (In *Proceedings,* Nat. Elec. Light Assoc., v. 77, pp. 526-529; Abstract, *Power,* v. 55, pp. 948-949.)
The type of intake rack, designed in two parts and ready for quick removal in case of sudden frazil ice, used at the Cedars Rapids hydro-electric plant in Quebec, is briefly described, but well illustrated on p. 528.

——**Stevens, J. C.** Winter Overflow from Ice Gorging on Shallow Streams. 1922. (In *Transactions*, Am. Soc. C. E., v. LXXXV (1922), pp. 677-696; Discussion, pp. 697-698.)
The facts gathered during the author's study of the winter overflow of the Madison River, Montana, are used as the basis of his paper on the occurrence, in the coldest part of the winter, of frazil ice in streams too turbulent to permit surface ice formation. The effects of this ice on bridges, piers, reservoirs, and channels are described, together with remedial measures.

——**Wegmann, E.** Design and Construction of Dams. Seventh Edition. 1922. (Wiley, N. Y., 555 pp.)
Quaker Bridge Dam is designed to resist an ice thrust of 43 000 lb. per lin. ft. (p. 160). Wachusett Dam will resist an ice pressure of 23.5 tons per lin. ft. (p. 194). Cross River Dam designed to resist ice pressure of 24 000 lb. per lin. ft.; Croton Falls Dam, of 30 000 lb. (pp. 203-204).

1923—Anderson, C. N. Ice Prevention on Trash Racks by Electric Heating. 1923. (In *Electrical World*, v. 82, p. 340.)
Illustrates racks adapted for electric heating at the Kykkelsrud Power Station, Norway.

——**Skerrett, R. G.** Reducing Ice Pressures with Compressed Air. 1923. (In *Canadian Engineer*, v. 45, pp. 181-185.)
Describes subaqueous jets of compressed air utilized to prevent ice formation on gates of the Mississippi River Power Co.'s plant at Keokuk, Iowa.

——**Walker, W. T.** Ice Problems of Hydro Plants. 1923. (In *Engineering and Contracting*, v. 60, pp. 1254-1264; Abstract of paper before American Society of Civil Engineers, 1923.)
Suggests methods of relieving gates of ice pressure; shows the importance and necessity of studying the effect of frazil, anchor, and shore ice on piers, canals, and hydro-electric structures.

1924—Bombing an Ice Jam. 1924. (In *Aviation*, v. 16, p. 459.)
Reports on the use of planes in dropping bombs on an ice jam in the Platte River, between Columbus and Valley, Nebr.

——**Effects of Ice on River Flow at Keokuk.** 1924. (In *Electrical World*, v. 83, p. 336.)
It is concluded that the flow during any winter will depend on the flow at the time of the first freeze-up. This initial freeze-up will reduce flow to 40% in 6 days and keep it at that for 5 days, after which 75 to 80% is gradually reached. Reasons given.

——**Freeman, John R.** General Review of Current Practice in Water Power Production in America. 1924. (In *Transactions*, First World Power Conference, v. 2, pp. 359-394.)
Discusses ice troubles and their cure (pp. 368-370).

——**Hogg, T. H.** Ice Problems at Hydro-Electric Plants. 1924. (In *Canadian Engineer*, v. 47, pp. 498-500.)
Frazil ice discussed, followed by a description of racks, ice skimmers, and immersion heaters. The specific ice problems of the St. Lawrence District are discussed.

——**Hollow Concrete Bridge Piers Built Like Chimneys.** 1924. (In *Engineering News-Record*, v. 93, p. 757; Discussion, v. 93, p. 924, and v. 94, p. 164.)
Ice pressure on piers of the Kentucky Hydro-Electric Co.'s Kennedy Bridge.

——**Hyatt, R. S.** Preventing Ice Formation on Water Turbines. 1924. (In *Electrical World*, v. 83, p. 386.)
Illustrates a successful arrangement of turbine parts, showing location of heating elements.

——**Kennedy, J. H.** Ice Control Methods at Shelburne Falls. 1924. (In *Power Plant Engineering*, v. 28, pp. 895-896.)
Describes the work which must constantly be carried on to keep the channels of the New England Power Co.'s Deerfield River plant in operation. Ice is held on either side of the channel in solid blocks by its attachment to booms. In this way the ice is melted slowly in the spring and a great deal of damage prevented.

——**Ruths, A.** Ice Troubles in Norwegian Water Power Plants. 1924. (In *Transactions*, First World Power Conference, v. 2, pp. 771-788; Abstract, *Engineering*, v. 118, pp. 73-74; also *Engineering and Contracting*, v. 62, pp. 805-809.)
A complete discussion of the nature of formation and relative danger of frazil, anchor, and sheet ice. Conditions existing at the Vamma Hydro-Electric Plant in Norway are described, together with means used to remove and prevent ice on turbines, gates, and at intakes.

——**Samsioe, A. F.** Measures Taken in Sweden Against Ice Troubles at Water Power Plants. 1924. (In *Transactions*, First World Power Conference, v. 2, pp. 806-811.)
Discusses the damage done by ice jams and ice sludge. Gives some figures on the amount of pressure exerted on dams by ice jams.

——**Westerlind, A.** Recent Practice in Swedish Water Power Engineering. 1924. (In *Transactions*, First World Power Conference, v. 2, pp. 338-358.)
Discusses the allowances made for ice pressure in designing dams, piers, and hydro-electric plants (pp. 341-344).

1925—Broadhead, A. P. Prevention of Damage from Anchor Ice. 1925. (In *Electrical World*, v. 86, p. 1108; also, *Engineering and Contracting*, v. 65, p. 53.)
Discusses the nature of formation and danger from such ice.

——**How to Blast Ice Jams.** 1925. (In *Engineering and Contracting*, v. 63, pp. 489-490.)
A 30 to 40% grade of gelatine dynamite is recommended for breaking ice on rivers. Directions are given for placing and firing the charge.

——**Ice Shove on High Bridge Piers.** 1925. (In *Engineering News-Record*, v. 93, p. 924; v. 94, pp. 164-165.)
L. F. Harza gives the resisting moment against overturning of the 22-ft. piers of the Kentucky Hydro-Electric Co.'s Kennedy Bridge, at 49 600 000 ft-lb. in a direction at right angles to the center line of the bridge and 30 800 000 ft-lb. in the direction of the bridge.

——**Loughland, G. E.** Ice Formations in the Main Sierra Canal. 1925. (In *Engineering and Contracting,* v. 63, pp. 775-780; copied from *Military Engineer.*)
Defines the different types of ice, conditions of formation, and methods of combatting them, in California.

——**Lowy, H. W.** Overcoming Ice Difficulties at the Holtwood Power Plant. 1925. (In *Journal,* Boston Soc. C. E., v. 12, pp. 142-149.)
Describes and illustrates the effects of river, forebay, and frazil ice on the operation of the Pennsylvania Water & Power Co.'s plant on the Susquehanna River. Statistics are given to show the power reduction due to frazil ice during 1919-1924.

1926—Barnes, H. T. Ice Conditions in St. Lawrence River. 1926. (In *Canadian Engineer,* v. 50, pp. 207-208.)
Discusses the effect of ice on water levels, the advantages provided by Nature, and describes a method worked out for the conservation of heat of Lake Ontario for ice prevention.

——**Carroll, E.** Guniting Up-Stream Face of Dam as Precaution Against Ice Damage. 1926. (In *Engineering News-Record,* v. 97, p. 432.)
Illustrates the application of gunite to protect Basin Creek Dam, a monolithic structure impounding the water supply of Butte, Mont. Surface ice on the reservoir caused severe damage to the dam face.

——**Dispositif d'Evacuation des Glaces et Corps Flottants dans les Usines à Turbines Verticales.** (Removal of Ice and Trash in Vertical Turbine Installations.) 1926. (In *Génie Civil,* v. 88, p. 237; also, *Power,* v. 63, p. 977.)
Illustrates a trash rack installed behind a vertical turbine to prevent débris from entering the turbine runner.

——**Franz, J.** Preventing Ice on Flash-Boards. 1926. (In *Electrical World,* v. 88, p. 1175.)
The Rockdale Hydro-Electric Station of the Southern Berkshire Power & Electric Co. has installed a galvanized iron pipe device connected with an air compressor which agitates the water and prevents freezing.

——**Ice Gorge Puts Power Plants Out of Commission.** 1926. (In *Power,* v. 63, p. 391.)
Announces the use of "thermite", a chemical developing a heat of 5 000° Fahr., in breaking up ice at Oil City, Pa.

——**Ice Jams Do Much Damage Along Allegheny River.** 1926. (In *Engineering News-Record,* v. 96, p. 503.)
Brief description of the destruction of three spans of the Big Rock Bridge, below Franklin, Pa., due to the breaking up of a severe ice jam.

——**Reid, C. R.** Overcoming Slush Ice Troubles. 1926. (In *Power,* v. 63, p. 977.)
Suggests means of preventing such ice entering the intake.

——**Removal of Ice and Trash in Vertical Turbine Installations.** 1926. (In *Power,* v. 63, p. 705.)
Gives diagram of proposed method of passing ice and trash through water power plant.

1927—Schwenkhagen, H. Bisherige Ergebnisse der Eislastforschung der Studiengesellschaft für Hochspannungsanlagen e. V. (Previous Results of Ice Pressure Research of the Studiengesellschaft for High-Tension Installation.) 1927. (In *Elektrizitätswirtschaft,* v. 26, pp. 10-18.)
Gives results of research on the formation and effect of ice on high-tension electric transmission lines.

DISCUSSION

C. M. DAILY,[19] M. AM. SOC. C. E. (by letter).—The conclusion of the Committee that ice troubles are "consolidated cold troubles" and that the heat in the water itself may be conserved by storage in protected, deep, stilled reservoirs, presents a logical method for lessening the quantity of "frazil" ice formed and the troubles due to it. The designing engineer's task seems to be only to exercise ingenuity in devising ways and means to accomplish and maintain this condition in his plant.

Another phase of trouble is presented by floating ice, and ice submerged by being weighted with silt in rapid streams and rivers which transport large quantities of sand both in suspension, and by rolling along their beds. This phase has caused the St. Louis Water Department to spend large sums during the thirty-six years since 1894 at its Chain of Rocks Plant, where two intake towers are located in mid-stream. These intakes, like bridge piers, cause violent swirls which carry the ice crushed against the tower into even the lowest ports—at times, more than 15 ft. below the surface of the water.

The swirls prevent bars from forming around the towers, as the sand is carried in suspension past them or into the ports. If it were not for sand-bars periodically passing the towers, the prevention of ice entering the ports would be comparatively simple. No solution of the ice trouble will be effected without giving due consideration to the vast quantity of débris, sand, boulders, and the like, which the Mississippi River playfully rolls along its bed or carries in suspension during periods of high water.

It is very evident that river intakes should be designed to create and maintain a deep and uniform flow of water past them at all stages, and that the ports should be placed far below the surface and be large enough to provide sufficient flow at a low velocity through them. A shore intake with suitable river-regulating works seems to offer the easiest and cheapest method for accomplishing this.

---

[19] Vice-Pres. and Chf. Engr., Missouri Eng. & Contr. Co., Inc., St. Louis, Mo.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

TRANSACTIONS

Paper No. 1789

STEEL COLUMN RESEARCH

SECOND PROGRESS REPORT OF THE SPECIAL COMMITTEE[1]

WITH DISCUSSION BY MESSRS. HENRY S. PRICHARD, E. H. SALMON, AND J. H. GRIFFITH

TO THE BOARD OF DIRECTION,
AMERICAN SOCIETY OF CIVIL ENGINEERS:

The study of column test data[2] which the Committee had partly completed and reported upon in January, 1926, made it appear desirable to conduct some experiments along certain lines on relatively small columns before completing its report. Accordingly, certain tests were planned and carried out during the past two years, which have yielded some results of value and which will be considered in this report.

It will be desirable, as a matter of convenience, to describe these tests and present the principal results before going on with the discussion in the order intended in the first report. This report, therefore, will consist of two parts: Part I is a description of the tests referred to, and Part II a continuation of the general report, as outlined in the Progress Report of 1926.

The tests described in Part I were made in the laboratories of the University of Wisconsin, under the direction of Professor M. O. Withey, of the Department of Mechanics, to whom the Committee is much indebted for the care and thoroughness with which this work has been done, and for the preparation of much of the material embodied in the report. Acknowledgment is also made of the valuable services of the following Research Assistants of the same Department: Messrs. R. R. Schrader, K. F. Wendt, E. A. Abendroth, R. T. Homewood, and E. R. Summers. Besides the use of its laboratories, the University of Wisconsin has contributed the test columns and considerable service by its Research Staff. The United States Forest Products Laboratory, through J. A. Newlin, M. Am. Soc. C. E., has assisted effectively by loaning the use of its 1 000 000-lb. testing machine and roller-bearing blocks. Engineering Foundation has co-operated in this work from the beginning, and a considerable part of the budget for 1927 has been secured through that organization.

---

[1] Presented at the Annual Meeting, January 16, 1929.

[2] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1485.

PART I.—TESTS OF COLUMNS MADE DURING 1925-27

1.—Object.—The principal object in mind in making these tests was to try out certain promising methods of studying the effectiveness of lacing-bars and other lateral elements. However, as the work developed, considerable valuable information was obtained regarding the stresses in such elements, the crookedness of certain columns, the relation of strength to yield point of material, the properties of the material at various points in a cross-section, the effect of form of section on eccentrically loaded columns, and other features. These various items will be discussed in their proper place after the tests have been described in general.

2.—Scope.—These column tests may be conveniently divided into three series, as follows:

(*A*) Eleven columns tested under oblique loading, causing a definite amount of shearing stress on the column.
(*B*) Ten rolled **H**-columns tested under eccentric loads.
(*C*) Two columns similar to certain columns of Series (*A*), tested under eccentric loads.

3.—Test Columns.—The general dimensions and properties of these columns are given in Tables 1 and 2. Details of columns, including location of gauge lines, are given in Figs. 1 to 7. Details of Columns Nos. 4, 5, 5*A*, 5*B*, and 6, which were latticed channel columns, are given in Fig. 4, and details of Columns Nos. 7, 8, and 9, which were batten-plate columns, are shown in Figs. 5, 6, and 7, respectively. Columns Nos. 4, 5, 5*A*, 5*B*, and 6 differ only in the size of the lacing-bars, those used in Column No. 4 being very light, and those in Column No. 6, extra heavy. Fig. 8 gives the locations of gauge lines for the **H**-columns of Series (*B*). Columns Nos. 5*A* and 8*A* of Series (*C*) were similar to Columns Nos. 5 and 8, respectively, of Series (*A*).

The actual sectional areas were computed from the weights, deducting the calculated weights of rivet heads, lacing-bars, and other details. The values of r were determined by measurement at each end of the column, and the two results were averaged.

All the columns listed in Tables 1 and 2 were contributed by the University of Wisconsin. All fabricated columns were manufactured by the American Bridge Company; and all **H**-columns were obtained from the Bethlehem Steel Company. The material and shopwork were specified to conform to the General Specifications for Railway Bridges of the American Railway Engineering Association.

4.—Apparatus.—Tests were made at the University of Wisconsin, some on a 1 000 000-lb. machine belonging to the United States Forest Products Laboratory, at Madison, and others with the 600 000-lb. testing machine in the Materials Testing Laboratory of the University. An important feature of the testing apparatus without which these tests could not have been readily made was the bearing-block arrangement placed between the ends of the column and the testing machine. These blocks were provided with rollers in

TABLE 1.—DIMENSIONS AND PROPERTIES OF COLUMNS OF SERIES (*A*) AND (*C*).

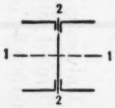
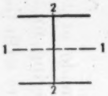
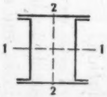
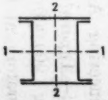
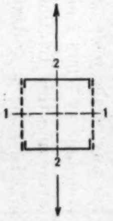
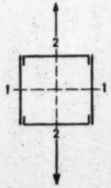
Column No. (1)	Type. (2)	Make-up. (3)	Length, in inches. (4)	AREA, IN SQUARE INCHES. Nominal. (5)	AREA, IN SQUARE INCHES. Actual. (6)	r_1, IN INCHES. Nominal. (7)	r_1, IN INCHES. Actual. (8)	$\frac{l}{r_1}$. (9)	I_2, in inches.⁴ (10)
1		5 by 3½ by 5/16-in. angles 10 by 5/16-in. plates	77	13.37	13.19	4.10	4.20	18.3	56.9
2		10-in., 49.5-lb. H-column	89.5	14.37	14.15	4.28	4.26	21.0	87.8
3		12 by ¼-in. plates 10-in., 15 lb. channels	89.5	14.92	14.67	4.42	4.43	20.2	210.9
3A		12 by ¼-in. plates 10-in., 15-lb. channels	89.5	14.92	14.43	4.42	4.42	20.3	188.8

TABLE 1.—(*Continued.*)

Column No.	Type.	Make-up.	Length, in inches.	Area, in Square Inches.		r_1, in Inches.		$\frac{l}{r_1}$.	I_2, in inches.[4]
				Nominal.	Actual.	Nominal.	Actual.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
4		10-in., 20-lb. channels 2¼ by 5/16-in. lacing-bars ¼ in. splice-plates	119	11.72	11.52	5.44	5.89	22.1	154.8
5		10-in., 20-lb. channels 2½ by ¼-in. lacing-bars ¼-in. splice-plates	119	11.72	11.30	5.44	5.44	21.9	151.4
5*A*		10-in., 20-lb. channels 2¼ by ¼-in. lacing-bars ¼-in. splice-plates	119	11.72	11.62	5.44	5.40	22.0	155.5
5*B*		10-in., 20-lb. channels 2¼ by ¼-in. lacing-bars ¼-in. splice-plates	119	11.72	11.38	5.44	5.44	21.9	152.6
6		10-in., 20-lb. channels 2½ by ⅜-in. lacing-bars ⅜-in. splice-plates	119	11.72	11.30	5.44	5.86	22.2	151.4
7		10-in., 20-lb. channels ¼-in. splice-plates Two 9 by 12 by ¼-in. batten-plates	119	11.72	11.84	5.44	5.44	21.9	158.7
8		10-in., 20-lb. channels ¼-in. splice-plates Four 9 by 12 by ¼-in. batten-plates	119	11.72	11.84	5.44	5.48	21.9	158.7
8*A*		10-in., 20-lb. channels ¼-in. splice-plates Four 9 by 12 by ¼-in. batten-plates	119	11.72	11.30	5.44	5.41	22.1	151.5
9		10-in., 20-lb. channels ¼-in. splice-plates Eight 9 by 12 by ¼-in. batten-plates	119	11.72	11.88	5.44	5.44	21.9	159.2

such a manner as to provide nearly frictionless end bearings to the columns. Fig. 9 shows a cross-section of one of the bearing blocks which explains the arrangement. Strain measurements over 8-in. and 20-in. gauge lengths were made with Berry strain-gauges read to 0.0001 in. For 2-in. gauge lines, an Olsen strain-gauge was read to 0.00003 in. Deflections were read by wire-wound dial deflectometers to 0.0001 in. The yield point was determined by the drop of the beam, and checked by the movement of the extensometer dial. The proportional limit was determined by the deviation point of the stress-strain curve from a straight line.

The frictionless character of the end bearings was indicated in various ways throughout the testing program, particularly by the sensitiveness of the column to small changes in position of the ends, and by the very close actual agreement of lateral deflection under eccentric loads with the theoretical, assuming hinged ends. However, in order to assure itself on this point, the Committee obtained direct evidence of the actual amount of the friction. The two blocks were placed face to face in the testing machine and a load applied directly to them. The combination of the two blocks was then turned through a considerable angle by means of a projecting rod attached to the blocks. Assuming that the center of curvature of the roller bearing of each block was precisely at the face of the block, the combination of the two could be turned through an angle with no resistance except friction. The maximum value of the load, *P*, was 500 000 lb., as compared to a maximum load used in the column tests of about 400 000 lb. It was found that up to a certain point the force required to turn the blocks increased with the amount of the movement, but beyond this point, the blocks continued to roll at very nearly constant force. The results of the tests, given in Table 3, are the averages of three to eight sets of readings. The friction coefficient is found by dividing the torque by the product of the diameter of the circle of roller centers times the imposed load, *P*. The angular movement of 0.00006 radians was equal to one division of the bubble used, which was the smallest movement noted.

In the oblique loading tests, it is found from calculation that the angle of rotation for Column No. 4, with $P/A = 14\,570$ lb. per sq. in. ($P = 167\,000$) and 5% shear, was about 0.002; and for Column No. 2, with $P/A = 14\,170$ lb. per sq. in. ($P = 200\,000$), it was about 0.0011. Comparing these values with the results given in Table 3 it would appear that the friction coefficient was in most cases considerably less than 0.002, and this may be taken as about the maximum probable value for any of the tests.

The effect of a friction coefficient of 0.002 can be judged by calculating the eccentricity necessary to overcome this friction. The frictional moment for a single bearing block would be $P \times 6 \times 0.002 = 0.012\ P$, requiring an eccentricity of only 0.012 in.; that is, the effective eccentricity would be 0.012 in. less than that given in the tests. Inasmuch as these values were of the magnitude of 1 in., or more, the error due to friction would be very small—generally less than 1 per cent.

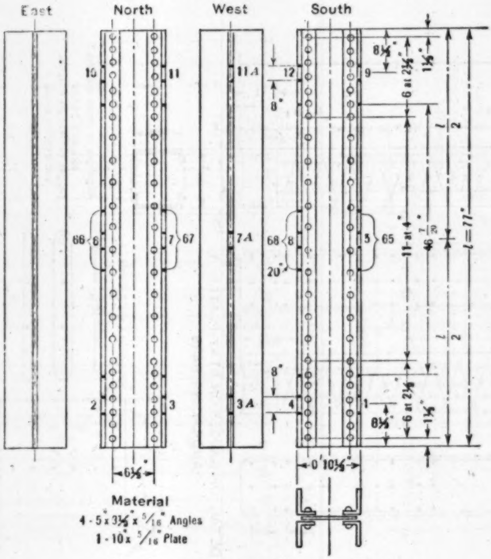

East
North
West
South
10
11
11A
12
9
8½"
6 at 2⅛"
1½"
8"
66
6
7
67
7A
68
8
5
65
20"
11 at 4"
46 7/20"
l/2
l = 77"
8"
2
3
3A
4
1
8½"
6 at 2⅛"
1½"
6½"
0' 10½"
Material
4 - 5" x 3½" x 5/16" Angles
1 - 10" x 5/16" Plate

FIG. 1.—DETAILS OF COLUMN NO. 1.

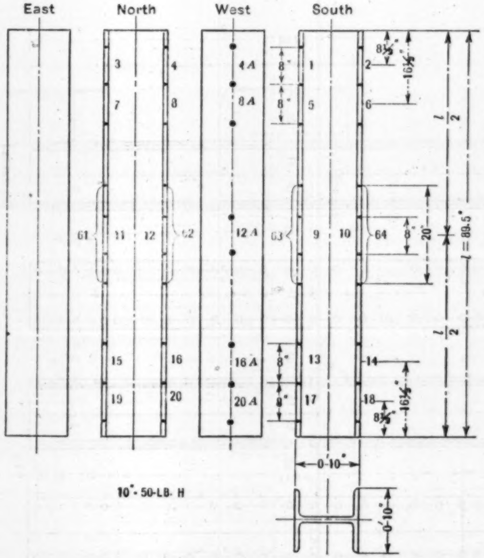

East
North
West
South
3
4
4A
8"
1
2
8½"
16½"
7
8
8A
8"
5
6
l/2
61
11
12
62
12A
63
9
10
64
8"
20"
l = 89.5"
15
16
16A
8"
13
14
l/2
19
20
20A
8"
17
18
16½"
8½"
0' 10"
10" - 50-LB. H
0' 10"

FIG. 2.—DETAILS OF COLUMNS NOS. 2 AND 2A.

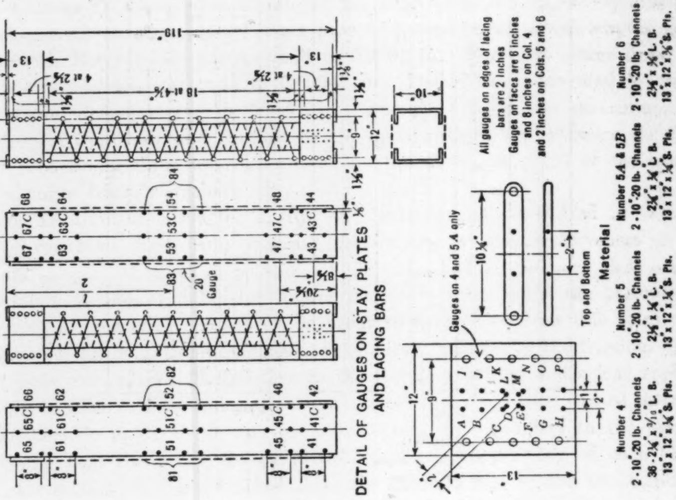

East
North
West
South
DETAIL OF GAUGES ON STAY PLATES AND LACING BARS
Gauges on 4 and 5A only
All gauges on edges of lacing bars are 2 inches
Gauges on faces are 6 inches and 8 inches on Col. 4
and 2 inches on Cols. 5 and 6
Top and Bottom
Material
Number 4
2-10" 20 lb. Channels
36-2¼" x 5/16" L. B.
13" x 12" x ⅜" S. Pls.
Number 5
2-10" 20 lb. Channels
2¼" x ¼" L. B.
13" x 12" x ⅜" S. Pls.
Number 5A & 5B
2-10" 20 lb. Channels
2¼" x ¼" L. B.
13" x 12" x ⅜" S. Pls.
Number 6
2-10" 20 lb. Channels
2¼" x ¼" L. B.
13" x 12" x ⅜" S. Pls.
119"
18 at 4½"
4 at 2¼"
13"
20" Gauge
10¼"

FIG. 4.—DETAILS OF COLUMNS NOS. 4, 5, 5A, 5B, AND 6.

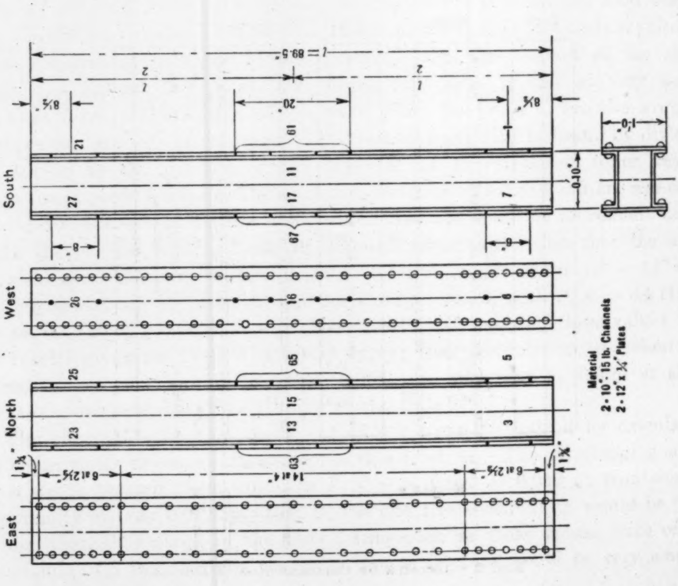

East
North
West
South
Material
2-10" 15 lb. Channels
2-12" x ¼" Plates
l = 89.5"
l/2
20"
14 at 4"
6 at 2¼"

FIG. 3.—DETAILS OF COLUMNS NOS. 3 AND 3A.

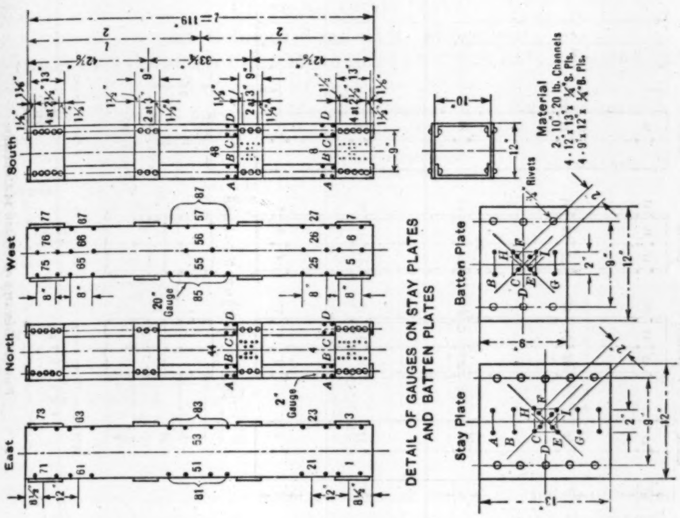

East
North
West
South
l = 119"
l/2
12⅝"
33¾"
DETAIL OF GAUGES ON STAY PLATES AND BATTEN PLATES
Stay Plate
Batten Plate
2" Gauge
20" Gauge
¾" Rivets
Material
2 - 10" - 20 lb. Channels
4 - 12" x 13" x ⅜" S. Pls.
4 - 9" x 12" x ⅜" B. Pls.

FIG. 6.—DETAILS OF COLUMN NO. 8.

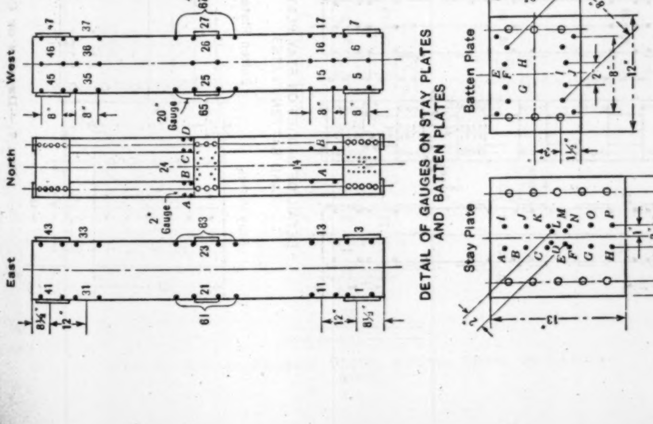

East
North
West
South
l = 119"
l/2
DETAIL OF GAUGES ON STAY PLATES AND BATTEN PLATES
Stay Plate
Batten Plate
2" Gauge
20" Gauge
Material
2 - 10" - 20 lb. Channels
4 - 12" x 13" x ⅜" S. Pls.
2 - 9" x 12" x ⅜" B. Pls.

FIG. 5.—DETAILS OF COLUMN NO. 7.

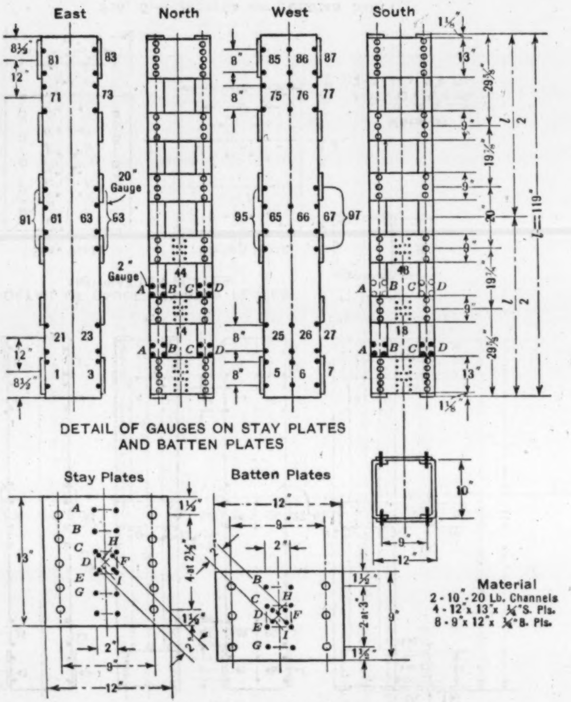
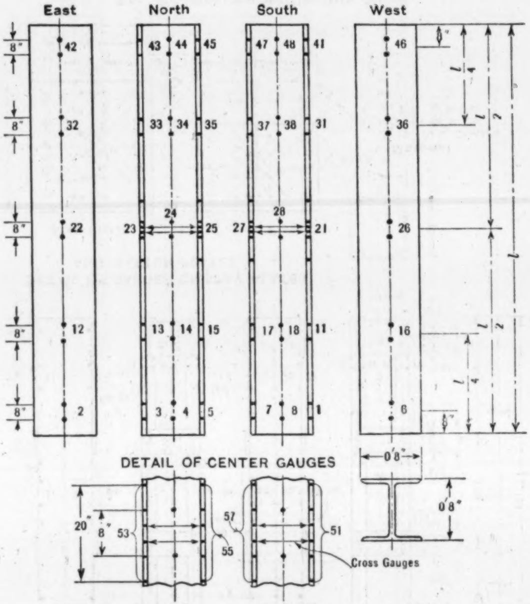

East
North
West
South
8½"
12"
81
83
71
73
20" Gauge
91
61
63
63
2" Gauge
44
A
B
C
D
14
21
23
12"
8½"
1
3
8"
85
86
87
75
76
77
95
65
66
67
97
25
26
27
5
6
7
1⅛"
13"
29⅝"
9"
19⅝"
20"
48
18
l/2
l = 119"
DETAIL OF GAUGES ON STAY PLATES AND BATTEN PLATES
Stay Plates
Batten Plates
13"
E
F
G
H
I
1½"
4 at 2⅛"
2"
9"
12"
2 at 3"
10"
Material
2 - 10" - 20 Lb. Channels
4 - 12" x 13" x ¼" S. Pls.
8 - 9" x 12" x ¼" B. Pls.

FIG. 7.—DETAILS OF COLUMN NO. 9.


East
North
South
West
8"
42
32
22
12
2
43
44
45
33
34
35
23
24
25
13
14
15
3
4
5
47
48
41
37
38
31
27
28
21
17
18
11
7
8
1
46
36
26
16
6
l/4
l/2
l
DETAIL OF CENTER GAUGES
20"
53
55
57
51
Cross Gauges
0'8"

NOTE:- Cross Flange Gauges are on Columns 1, 2, 3, 4, 6, 7 & 8

Columns No.	Length Inches
1 & 6	39.5
2 & 7	99.0
3 & 8	168.2
4 & 9	237.5
5 & 10	306.9

FIG. 8.—DETAILS OF 8-INCH H-COLUMNS.

TABLE 2.—DIMENSIONS AND PROPERTIES OF COLUMNS OF SERIES (*B*) TESTED UNDER ECCENTRIC LOADS.

All Columns 8-in., 32-lb. **H**-Sections.

(Nominal Values: Area = 9.17 sq. in.; $I_1 = 105.7$ in.^{4}; $I_2 = 35.8$ in.^{4}; $r_1 = 3.40$ in.; $r_2 = 1.98$ in.

Gravity Axis 1-1, parallel to flanges; Gravity Axis 2-2, through web.)

Column No.	Length, in inches.	Area, in square inches.	Radii of Gyration, in Inches.		Slenderness Ratios.	
			r_1.	r_2.	$\frac{l}{r_1}$.	$\frac{l}{r_2}$.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
H 1	39.1	9.40	3.42	1.95	11.4	20.1
H 6	39.6	9.61	3.44	1.95	11.5	20.3
H 10 *B*	39.6	8.96	3.42	1.94	11.6	20.4
H 2	99.0	9.37	3.42	1.95	29.0	50.8
H 7	99.0	9.63	3.43	1.96	28.9	50.6
H 3	168.3	9.28	3.40	1.94	49.5	86.8
H 8	168.3	9.63	3.44	1.95	48.9	86.3
H 4	237.8	9.45	3.41	1.94	69.6	122.4
H 9	237.5	9.32	3.40	1.97	69.9	120.5
H 5	306.9	8.97	3.42	1.95	89.7	157.3
H 10	306.9	8.92	3.42	1.94	89.7	158.2

The question has also been raised whether or not in the oblique load tests it could be safely assumed that the applied load is always truly vertical, or whether a horizontal component of considerable magnitude may not be developed as the column becomes distorted, thus giving rise to considerable error in the calculated shear. Inasmuch as the bearing blocks were not free to move laterally, any tendency of the column to take on a horizontal movement of one end with respect to the other would be resisted by the testing machine and thus give rise to a horizontal load component. The analysis of the various kinds of distortion to which the column is subjected—flexure, shear, and direct stress—indicates that such possible effect is very small; but probably the best answer is a comparison of the observed bending stresses (calculated from strain-gauge readings) near the ends of the column, with the values calcu-

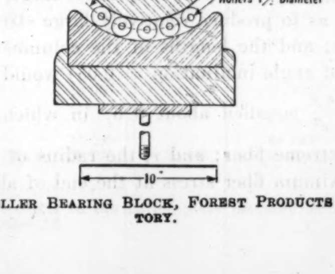


FIG. 9.—ROLLER BEARING BLOCK, FOREST PRODUCTS LABORATORY.

lated from the assumed shear. Such comparison has been made for four columns under an oblique load assumed to give 5% shear (obliquity, 0.05). For a total of seven different loads, the maximum difference between the two methods of calculation, averaging results for both ends, was 14 per cent. Most values were within 3%, some being of one sign and some of the other. The average of the seven was 2.7%, the strain-gauges giving the higher value. Considering the nature of strain-gauge work, this agreement is quite satisfactory.

TESTS OF SERIES (*A*): OBLIQUE LOADING

5.—Procedure in Testing.—These tests were undertaken to secure information regarding lateral rigidity and strength, but more especially to try out a certain method of testing which it was thought would be particularly well adapted for the study of the various problems connected with lateral strength.

The question of shearing stress and shearing strength of columns is a very important one, and one concerning which the usual test with a central load gives little information. In order to impose a relatively large amount of shearing stress upon these columns, they were loaded obliquely, that is, by placing the load eccentrically at the end, the eccentricities at the two ends being in opposite directions. The bearing blocks were very nearly frictionless, and their behavior indicated that the columns were substantially pivoted end columns. By this arrangement, a definite amount of shearing stress in proportion to the direct stress could be imposed. Fig. 10 shows the manner of placing the column in the testing machine.

TABLE 3.—ROLLING FRICTION OF BEARING BLOCKS.

Load, *P*.	ANGULAR MOVEMENT OF 0.00006 RADIANS.		ANGULAR MOVEMENT OF 0.0006 RADIANS.		CONTINUOUS ROLLING.	
	Torque, in foot-pounds.	Friction coefficient.	Torque, in foot-pounds.	Friction coefficient.	Torque, in foot-pounds.	Friction coefficient.
100 000	1 610	0.00134	2 380	0.00198	3 040	0.00253
200 000	2 000	0.00083	3 040	0.00126	5 020	0.0021
300 000	2 420	0.00064	4 900	0.00136	9 100	0.0024
400 000	2 760	0.00058	6 330	0.00132	16 800	0.0035
500 000	2 550	0.00043	7 400	0.00123	......	....

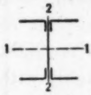
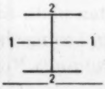
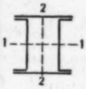
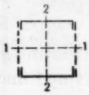
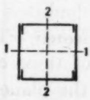
Strain-gauge measurements were made at all corners near the ends, quarter-points, and center, on 8-in. gauge lengths, and also at the center on 20-in. gauge lengths. Strain measurements were also made on lacing-bars, stay-plates, and batten-plates. The locations of the gauge lines are shown in Figs. 1 to 7. The shearing stress was purposely made large, 5% of the direct stress for final loading, so as to produce relatively large stresses in the lateral connections and details; and the lengths of the columns were chosen so that a 5% shear (tangent of angle inclination = 0.05) would require an eccentricity at the end such that $\frac{e\,c}{r^2}$ equalled about 0.6, in which, e is the eccentricity; c, the distance to extreme fiber; and r, the radius of gyration. Such eccentricity caused a maximum fiber stress at the end of about 1.6 times the average, or 1.6 times P/A. In all cases, the plane of obliquity was parallel to Axis 2-2 (Table 1), the roller axes being parallel to Axis 1-1.

The columns were first centered in the testing machine by measurement and then by trial loadings until the strain-gauge measurements and deflection apparatus showed practically no eccentricity under a considerable load. After being centered, the ends were shifted a calculated amount to produce the desired obliquity.

Some of the columns were tested under 2% shear up to a maximum fiber stress of about 14 000 lb. per sq. in., and then tested to failure under 5% shear. Others were tested only under 5% shear.

6.—Maximum Stresses at Ultimate Load.—Table 4 gives the results of these tests relative to the ultimate loads and the maximum fiber stresses at the ends of the columns.

TABLE 4.—ULTIMATE LOADS AND MAXIMUM FIBER STRESSES AT ENDS OF COLUMNS UNDER 5% SHEAR.

Column No.	Type.	APPLIED LOADS, $\frac{P}{A}$, IN POUNDS PER SQUARE INCH AT:		CALCULATED FIBER STRESSES AT ENDS, IN POUNDS PER SQUARE INCH AT:		AVERAGE TENSILE PROPERTIES OF MATERIAL, IN POUNDS PER SQUARE INCH.	
		First scaling.	Ultimate.	First scaling.	Ultimate.	Yield point.	Proportional limit.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1		22 000	28 000	34 500	43 900	38 300	33 500
2		19 500	26 700	31 400	42 900	36 100	33 700
3		18 000	25 900	28 900	41 500	35 000	33 200
3*A*		18 000	24 900	28 800	39 900	36 500	33 400
4		18 000	22 800	29 000	36 800	32 000	26 400
5		18 600	22 900	29 900	36 800	32 000	26 400
5*A*		22 000	22 300	35 400	35 900	32 300	30 000
6		17 500	24 000	28 200	38 700	32 000	26 400
7		......	10 000	......	16 000*	32 700	27 400
8		......	15 000	......	24 100*	32 200	28 400
9		19 000	20 700	30 500	33 200*	31 700	27 800

* The maximum unit stresses in the batten-plate columns occurred in the unsupported channels near plates, see Table 13.

Columns Nos. 1, 2, 3, and 3*A*, composed of solid rolled or riveted sections, failed in the manner indicated in Fig. 10. The latticed channel columns also failed by over-stressing the channels, except Column No. 4 with the light lacing, which failed both by over-stressing of the channels and by the buckling of the lacing. This is shown very clearly in Figs. 11 and 12. In all these columns, however, the calculated maximum fiber stress at the ends was much beyond the yield point of the material. As Table 4 shows, the first scaling occurred in some cases before the average unit stress on the extreme fiber reached the recorded proportional limit. The batten-plate columns failed as indicated in Fig. 13, as would be expected. The excessive shearing stress caused large bending moments in the individual channels, which led to failure at the first rivet of the end batten-plate. Column No. 9, however, carried an ultimate load almost as great as the maximum sustained by the latticed channel columns.

7.—*Shearing Rigidity and Strength.—Transverse Bending Tests.*—Before testing the columns as columns, they were tested as simple beams under center loads up to calculated fiber stresses of 1 000 to 6 000 lb. per sq. in. By comparing the deflections and sets when bent in different directions, some information was obtained relative to the effectiveness of lateral connections. Table 5 gives important results of these tests.

All the columns when tested with the plane of the load parallel to the web exhibited small sets and the deflections agree reasonably well with calculated values if due allowance is made for shear deflection. When tested with the plane of loading normal to the web, that is, parallel to the lacing or battens, both the laced channel columns, Nos. 4 to 6, and the batten-plate columns, Nos. 7 to 9, showed considerable set as regards deflections after light loads. The elastic deflections of the laced columns so tested agree roughly with calculations, including an allowance for shear deflection. For equal fiber stresses and span lengths, they were approximately equal. The batten-plate columns exhibited much higher deflections. Under this type of loading, basing the comparison on the same fiber stresses, the elastic deflections of Columns Nos. 7, 8, and 9 were, respectively, seven, three, and two times as much as the corresponding deflections of the laced columns.

It is of interest to compare the observed deflections with the calculated values, considering bending distortion only. Table 6 gives some results of such calculation. The calculated values are conveniently obtained from the formula,

$$\text{Deflection} = \frac{1}{6}\,\frac{f\,l^2}{E\,h}$$

in which, f is the bending fiber stress and h, the depth of the beam in the plane of bending. When bent in the plane of the webs, the solid riveted and rolled sections showed a deflection somewhat more than the calculated; the laced channel columns, somewhat less. This smaller deflection is doubtless due to the stiffening effect of lacing and stay-plates, which was neglected. The important result is, however, in the relatively large deflections of these columns (from 1.6 to 2.0 times the calculated values) when bent in the plane of the lacing. This excess is due obviously to the contribution of the lacing and

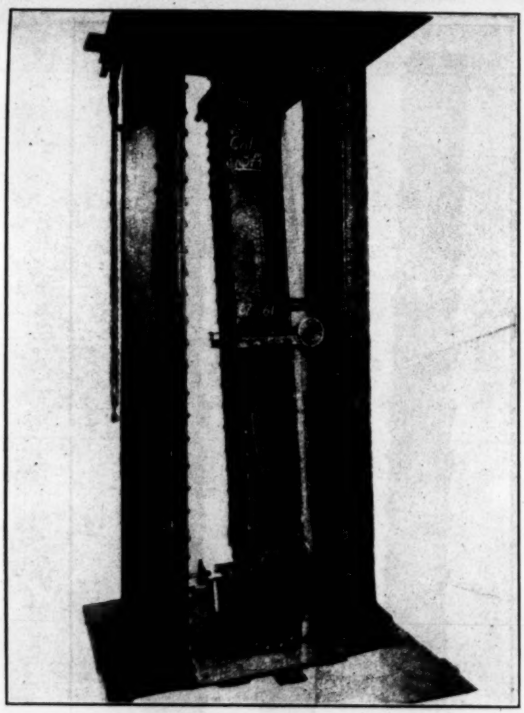


FIG. 10.—APPARATUS USED TO MEASURE SHEAR DISTORTIONS AND DEFLECTIONS IN OBLIQUE LOADING TESTS. NOTE, ALSO, METHOD OF FAILURE OF COLUMN NO. 3A.

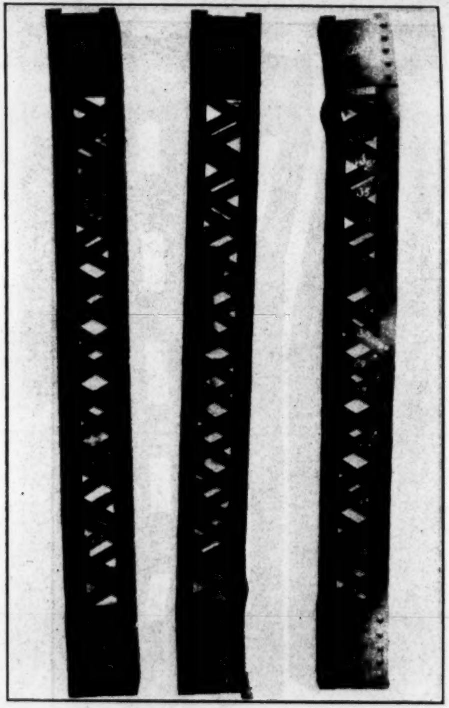


FIG. 11.—FAILURES OF COLUMNS NOS. 4, 5A, AND 6, TESTED UNDER OBLIQUE LOADS.

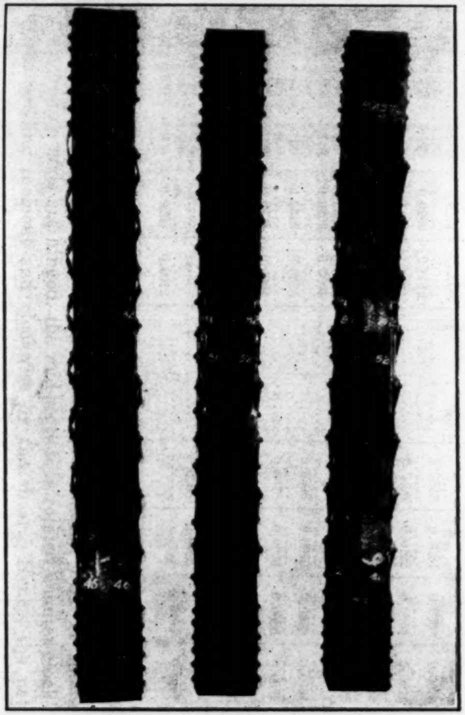


FIG. 12.—FAILURES OF COLUMNS NOS. 4, 5A, AND 6 TESTED UNDER OBLIQUE LOADS. NOTE BUCKLING OF LACING-BARS ON COLUMN NO. 4.

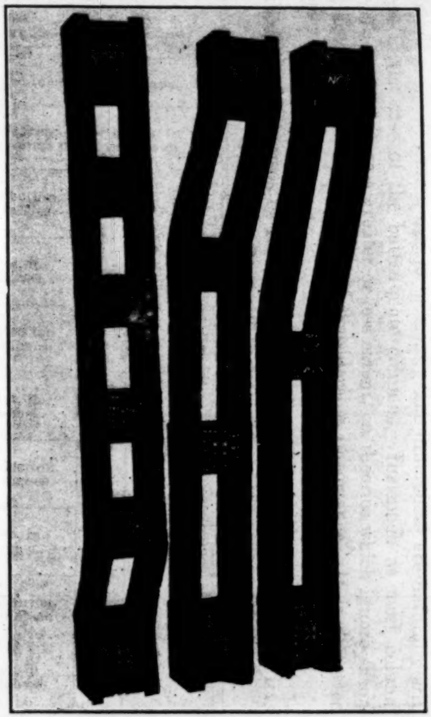


FIG. 13.—FAILURES OF COLUMNS NOS. 7, 8, AND 9.

stay-plates to the deflection, or, as it may be considered, the deflection due to shear; that is, the deflection due to shear is, in this case, from 60 to 100% as much as the deflection due to moment. The differences are naturally still larger in the case of the batten-plate columns. Inasmuch as most column formulas are based, to a greater or less extent, on theories which ignore shearing distortion, this question is of considerable importance. It is discussed further in Part II.

TABLE 5.—COMPARISON OF DEFLECTIONS AND SETS AT MID-LENGTH OF COLUMNS UNDER SMALL TRANSVERSE LOADS.

Column No.	Length of span, in inches.	Center load, in pounds.	Extreme fiber stress, in pounds per square inch.	Position of Load Plane.							
				Parallel to Web.				Normal to Web.			
				Direct.		Inverted.		Direct.		Inverted.	
				Deflection, in inches.	Set, in inches.	Deflection, in inches.	Set, in inches.	Deflection, in inches.	Set, in inches.	Deflection, in inches.	Set, in inches.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	72.0	7 320	2 950	0.0118	0.0000	0.0118	0.0000	……	……	……	……
2	84.0	7 620	3 060	0.0148	−0.0006	0.0146	−0.0005	……	……	……	……
3	84.0	8 000	3 100	0.0169	*0.0035	……	……	……	……	……	……
		7 920	3 070	……	……	0.0135	0.0005	……	……	……	……
3A	84.0	9 820	3 820	0.0169	0.0002	0.0144	0.0000	……	……	……	……
4	114.0	8 300	3 060	0.0197	0.0002	……	……	……	……	……	……
		6 120	3 120	……	……	……	……	0.0358	0.0084	0.0367	0.0079
5	114.0	3 300	3 120	0.0203	0.0012	……	……	……	……	……	……
		6 120	3 130	……	……	……	……	0.0406	0.0122	0.0364	0.0075
5A	117.5	6 290	3 260	……	……	……	……	0.0402	0.0060	……	……
		5 780	2 980	……	……	……	……	……	……	0.0387	0.0068
		4 200	3 980	0.0294	0.0003	……	……	……	……	……	……
5B	117.5	9 480	4 920	……	……	……	……	0.0496	0.0023	0.0531	0.0075
		6 160	5 940	0.0433	0.0008	……	……	……	……	……	……
6	114.0	3 320	3 140	0.0209	0.0001	……	……	……	……	……	……
		6 120	3 180	……	……	……	……	0.0306	0.0070	0.0320	0.0061
7	117.5	4 320	4 000	0.0325	0.0025	……	……	……	……	……	……
		1 980	1 000	……	……	……	……	0.0678	0.0024	0.0688	0.0029
8	117.5	4 300	4 000	0.0278	0.0004	……	……	……	……	……	……
		3 980	2 010	……	……	……	……	0.0600	0.0041	……	……
		4 000	2 020	……	……	……	……	……	……	0.0645	0.0065
8A	117.5	6 170	6 000	0.0168	0.0010	……	……	……	……	……	……
		3 750	1 990	……	……	……	……	0.0628	0.0056	0.0628	0.0070
9	117.5	6 000	3 020	……	……	……	……	0.0505	0.0055	……	……
		5 970	3 000	……	……	……	……	……	……	0.0496	0.0061
		4 340	4 000	0.0293	0.0012	……	……	……	……	……	……

*Member did not have full bearing across knife-edges due to initial wind of approximately 3⅛ degrees.

8.—Oblique Loading Tests.—As regards oblique loading, Fig. 14 shows that, if the column ends are hinged, the mid-section is rotated counter-clockwise by the bending moment and clockwise by the shear. Hence, the net angular rotation of the mid-section, α, relative to the column axis is the difference of the shear and bending rotations, $\alpha_S - \alpha_B$. Also, owing to small rotations of the bearings and the machine during loading, the column as a whole suffers a small rotation, α'. Hence, the entire rotation of the mid-section, α_1, will be $\alpha + \alpha'$. By attaching a pair of spirit-levels to a side-bar supported at the column ends (Fig. 10) the rotation of the column as a whole, α', was measured. Spirit-levels, termed "shear-minus-moment" levels in Fig. 14, attached at the mid-section measured the entire rotation of that section, α_1. Then,

$$\alpha_S - \alpha_B = \alpha_1 - \alpha'$$

TABLE 6.—COMPARISON OF OBSERVED AND CALCULATED DEFLECTIONS, CROSS-BENDING TESTS.

Column No.	Length of span, in inches.	Depth of section, h, in inches.	Extreme fiber stress, f, in pounds per square inch.	BENDING PARALLEL TO WEB.			BENDING PARALLEL TO LACING PLANE.		
				Observed deflection, A, in inches.	Calculated deflection, B, in inches.	Ratio, $\frac{A}{B}$.	Observed deflection, A, in inches.	Calculated deflection, B, in inches.	Ratio, $\frac{A}{B}$.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	72.0	10.38	2 950	0.0118	0.0085	1.39	......	......	......
2	84.0	9.84	3 060	0.0147	0.0126	1.17	......	......	......
2 *A*	84.0	10.46	3 820	0.0157	0.0148	1.06	......	......	......
4	114.0	10.00	3 060	0.0197	0.0229	0.86	......	......	......
		11.90	3 120	......	......	......	0.0358	0.0196	1.83
		11.90	3 120	......	......	......	0.0367	0.0196	1.87
5	114.0	10.00	3 120	0.0203	0.0233	0.87	......	......	......
		12.00	3 130	......	......	......	0.0406	0.0195	2.08
		12.00	3 130	......	......	......	0.0364	0.0195	1.87
5 *A*	117.5	10.00	3 980	0.0294	0.0315	0.93	......	......	......
		11.94	3 260	......	......	......	0.0402	0.0216	1.86
		11.94	2 980	......	......	......	0.0387	0.0198	1.95
5 *B*	117.5	9.99	5 940	0.0433	0.0472	0.92	......	......	......
		12.03	4 920	......	......	......	0.0496	0.0324	1.53
		12.03	4 920	......	......	......	0.0531	0.0324	1.64
6	114.0	10.00	3 140	0.0209	0.0234	0.89	......	......	......
		11.82	3 180	......	......	......	0.0306	0.0201	1.52
		11.82	3 180	......	......	......	0.0320	0.0201	1.59
7	117.5	10.04	4 000	0.0325	0.0316	1.03	......	......	......
		12.02	1 000	......	......	......	0.0678	0.0066	10.28
		12.02	1 000	......	......	......	0.0688	0.0066	10.41
8	117.5	10.04	4 000	0.0278	0.0316	0.88	......	......	......
		12.00	2 010	......	......	......	0.0600	0.0133	4.51
		12.00	2 020	......	......	......	0.0645	0.0133	4.85
8 *A*	117.5	10.00	6 000	0.0468	0.0480	0.98	......	......	......
		11.93	1 990	......	......	......	0.0628	0.0132	4.76
		11.93	1 990	......	......	......	0.0628	0.0132	4.76
9	117.5	10.03	4 000	0.0293	0.0316	0.93	......	......	......
		12.01	3 020	......	......	......	0.0505	0.0199	2.54
		12.01	3 000	......	......	......	0.0496	0.0198	2.50

In most of the tests, α_B was calculated from the equation,

$$\alpha_B = \frac{0.05 \dfrac{P\,l^2}{24\,E\,I}}{1 - \dfrac{P\,l^2}{24\,E\,I}} \quad \ldots\ldots\ldots\ldots \quad \ldots\ldots\ldots\ldots\ldots (1)^3$$

---

[3] From "Modern Framed Structures," Part 2, p. 508, by Johnson, Bryan and Turneaure, the general equation of the elastic curve of such a column is found to be,

$$y = c_1 \sin q\,x + c_2 \cos q\,x - \frac{M_1}{P}\,\frac{V_1\,x}{P}$$

in which, $q = \sqrt{\frac{P}{E\,I}}$ and M_1 and V_1 are the moment and shear, respectively, at the end taken as origin. By differentiation the constants may be eliminated and the rotation of the mid-section can be determined from the first derivative of the equation for y. By substituting $e = 0.025l$ and $V_1 = 0.05$,

$$\alpha_B = 0.05 - \frac{0.05\,q\,\frac{l}{2}}{\sin q\,\frac{l}{2}}$$

Expansion of the sine term reduces this to Equation (1).

and the net angular shear distortion, α_S, was determined from the equation,

$$\alpha_S = \alpha - \alpha' + \alpha_B$$

In tests on Columns Nos. 2*A* and 5*B* the shear distortion was measured more directly by adding rotations measured by "shear-minus-moment" levels to those measured by the "moment" levels (see Fig. 14).

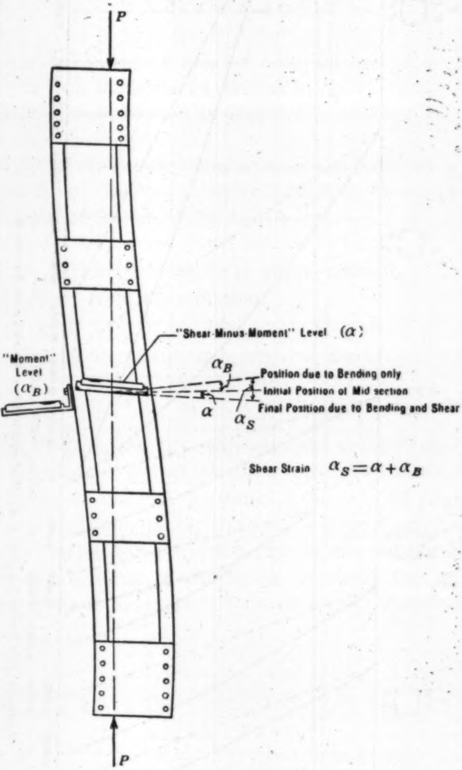

P
"Shear-Minus-Moment" Level (α)
"Moment" Level (α_B)
α_B
Position due to Bending only
Initial Position of Mid section
Final Position due to Bending and Shear
α
α_S
Shear Strain $\alpha_S = \alpha + \alpha_B$
P

FIG. 14.—METHOD OF MEASURING SHEAR STRAINS IN COLUMNS OF SERIES (*A*).

Typical results of these shear distortion measurements are shown in Fig. 15 for unit loads, P/A, up to 15 000 lb. per sq. in. The diagram shows sets after certain loads as well as deformations under load. The solid rolled and riveted sections show little shear distortion and negligible set; the laced channel columns, considerable shear distortion and relatively large sets; and the batten-plate columns, very large distortions and sets.

In the case of the batten-plate columns the shear distortion is, in part, local bending of channels between plates, but the result is a measure of shear flexibility as a whole and of set. The superior rigidity of the solid and riveted sections is marked.

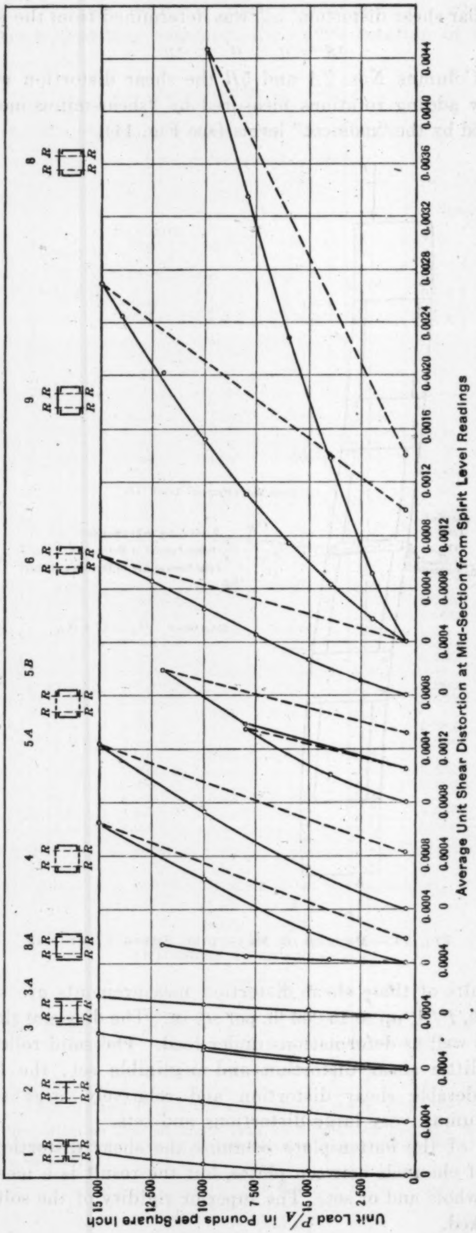


FIG. 15.—UNIT LOAD SHEAR DISTORTION CURVES FOR COLUMNS TESTED UNDER OBLIQUE LOADS. SHEAR 5 PER CENT.

8(a).—Comparison of Computed and Observed Shear Distortions.—Calculations of the unit shear distortions of these columns were also made.

Solid Rolled and Riveted Sections.—For these sections the distortions were computed as follows:

$$\alpha_S'' = \frac{V}{E_S\, I\, d} \sum \frac{A'\, y'}{b}\, d\, y \qquad \text{(2)}$$

in which,

$A'y'$ = statical moment of area of cross-section above *m-m* (shaded in Fig. 16) about the neutral axis;

b = width of cross-section at *m-m*;

d = depth of section;

$d\,y$ = height of elementary lamina under consideration;

E_S = modulus of elasticity in shear, 11 600 000 lb. per sq. in.;

I = moment of inertia about Axis 1-1;

V = the total transverse shear on the column.

Laced Channel Sections.—The unit shear distortion for laced channel sections was calculated from the equation,

$$\alpha_S'' = \frac{2\, K_1\, V\, l_1}{L\, E_S\, I_1^2} + \frac{V\,(l_0^3\; l_2^3)}{12\, L\, E\, I_2} + \frac{K_2\, V\,(l_0 + l_2)}{L\, E_S\, I_2^2} + \frac{n\, S\, u\, l}{L\, A_1\, E}$$

$$+ \frac{2\, n\, l_3\, V \tan^2{}_\theta}{A_3\, E}\left(\frac{25}{54} + \frac{l_3^2}{40\, r_3^2}\right) \qquad \text{(3)}$$

The first term of Equation (3) represents the shear distortion of the stay-plate portion[4]; the second and third terms give the bending and shear distortions, respectively, of the four portions of the column having unsupported channels adjoining each stay-plate; the fourth term furnishes the distortion due to straining of lacing-bars; and the fifth represents the distortion caused by the shearing and bending of the rivets fastening the lacing-bars to the channels. Each rivet was assumed to be a uniformly loaded cantilever.

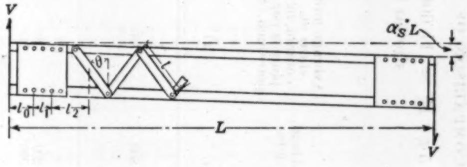


FIG. 17.

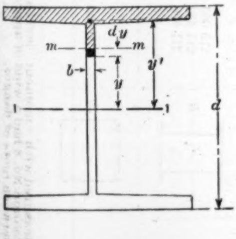


FIG. 16.

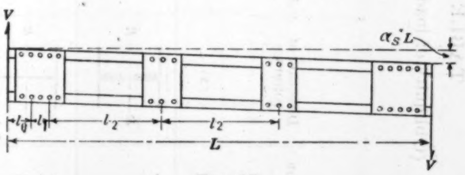


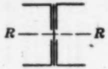
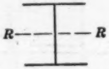
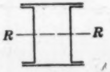
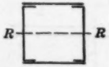
FIG. 18.

---

[4] The derivation of the general expression for shear deflection may be found in "Strength of Materials," by Maurer and Withey, p. 322.

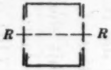
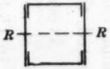
TABLE 7.—COMPARISON OF THE SHEARING RIGIDITIES OF THE COLUMNS OF SERIES (*A*):

(Obliquity of loading, 5%; E_s = 11 600 000 lb. per sq. in. Columns Nos. 1, 2, 3, 4, and 6 had been subjected to similar loading prior to the runs herein recorded.)

Column No.	Direction of rollers.	Applied load, in pounds.	Average unit shear on column, in pounds per square inch, (S).	Modulus of elasticity, 1 000 000 lb. per sq. in.	AVERAGE UNIT SHEAR DISTORTION.				Relative rigidity, $\left(\frac{S}{10\ 000\ a_s'}\right)$.
					Measured.		Calculated, (a_s'').	Ratio, $\left(\frac{a_s'}{a_s''}\right)$.	
					Total, (a_s).	Elastic, (a_s').			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1		192 800	730	28.2	0.000188	0.000188	0.000155	1.21	388
2 2 *A*		207 200 220 000	733 748	29.7 29.0	0.000241 0.000279*	0.000241 0.000271*	0.000253 0.000258	0.95 1.05	304 276
3 3 *A* 3 *A*		215 000 211 450 211 450	734 734 734	28.6 27.4 27.4	0.000277† 0.000188‡ 0.000157	0.000277† 0.000188‡ 0.000157	0.000161 0.000190‡ 0.000161	1.72 0.99 0.98	264 390 467
8 *A*		130 600	578	29.9	0.000072	0.000072	0.000072	1.00	803

* Measured with "moment" level apparatus.
† Column No. 3 had a wind of approximately 8½ degrees.
‡ Between roots of flanges.

TABLE 7.—(*Continued.*)

Column No.	Direction of rollers.	Applied load, in pounds.	Average unit shear on column, in pounds per square inch, (S).	Modulus of elasticity, 1 000 000 lb. per sq. in.	AVERAGE UNIT SHEAR DISTORTION.				Relative rigidity, $\left(\frac{S}{10\ 000\ a_s'}\right)$.
					Measured.		Calculated, (a_s'').	Ratio, $\left(\frac{a_s'}{a_s''}\right)$.	
					Total, (a_s).	Elastic, (a_s').			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
4		167 800	728	29.0	0.001046	0.000863	0.000848	1.02	84
5 *A*		169 300	728	29.0	0.001232	0.000784	0.000702	1.12	93
5 *B*		181 320	577	29.4	0.001022*	0.000489*	0.000555	0.88	118
5 *B*		181 320	577	29.4	0.000731	0.000478	0.000555	0.86	121
5 *B*		40 400	177	29.4	0.000161	0.000141	0.000171	0.82	125
6		164 500	728	29.0	0.001044	0.000644	0.000509	1.26	113
7		89 700	379	29.0	0.007399	0.005229	0.004740	1.10	7
8		113 400	479	29.0	0.004448	0.003041	0.002770	1.10	16
8 *A*		96 700	428	29.9	0.003521	0.002611	0.002450	1.07	16
8 *A*		40 200	178	29.9	0.001433	0.001073	0.001020	1.02	17
9		173 200	730	29.0	0.002711	0.001734	0.001610	1.08	42

* Measured with "moment" level apparatus.

The significances of the undefined terms are:

A_1 = cross-sectional area of a lacing-bar;

A_3 = cross-sectional area of a rivet in the lacing-bar;

I_1 = moment of inertia of stay-plate portion about a gravity axis parallel to channel webs;

I_2 = least moment of inertia of a single channel about its own gravity axis;

$K_1 = \int \frac{(A' \, y')^2 \, d \, y}{b}$, a constant for the stay-plate cross-section;

K_2 = constant like K_1 for a single channel;

l, l_0, l_1, l_2, and L are defined in Fig. 17;

l_3 = the cantilever length of rivet which was taken equal to twice the thickness of a lacing-bar plus one-half the rivet grip in the channel flange;

n = number of lacing-bars in the column;

r_3 = radius of a rivet;

$S = \frac{V}{2} \sec \theta$;

$u = \frac{1}{2} \sec \theta$.

Batten-Plate Columns.—For these columns (Fig. 18), the unit shear distortion was calculated from,

$$\alpha_S'' = \frac{K_2 \, V \, l_0}{L \, E_S \, I_2^2} + \frac{V \, l_1^3}{12 \, L \, E \, I_2} + \frac{K_1 \, V \, l_1}{L \, E_S \, I_1^2} + \frac{K_2 \, V \, n \, l_2}{2 \, L \, E_S \, I_2^2} + \frac{V \, n \, l_2^3}{24 \, L \, E \, I_2} \cdots (4)$$

In Equation (4) the first two terms give the shear and bending distortion, respectively, of the two extreme end portions, l_0 in length; the third term furnishes the shear distortion of the two stay-plate portions, each l_1 in length; and the last two terms, which are of major importance, furnish the shear and bending distortion, respectively, of the several (n) portions of the column, each having a length, l_2, and containing two unsupported channels. The other symbols have been previously defined.

Comparison of Shear Distortions.—The relation between the measured and calculated shear distortions for the columns of Series (A) is given in Table 7. Aside from Column No. 3, which was defective, it will be observed from Column (9) of Table 7, that there is good agreement between the elastic measured unit shear distortions and the values calculated by Equations (1) to (4).

To afford a comparison of the relative shearing rigidities of the various types of columns tested, the ratios of,

$$\frac{\text{Average unit shear stress on column}}{\text{Measured elastic unit shear distortion}}$$

have been entered in Column (10) of Table 7. As would be expected from consideration of Equation (2), Column No. 8 A (with rollers normal to webs) and the riveted box sections exhibited the highest shearing rigidity. The superior shearing rigidity of the laced-channel columns over the batten-plate columns is also noteworthy.

9.—*Stresses in Lacing.*—The tests on laced-channel columns afforded a good opportunity to examine the stresses in the lacing-bars. These were determined both by calculation[5] and by strain measurement for the central and also for the oblique loading tests. At first, the strain measurements were made on the median lines of the outer faces of the bars using 6 or 8-in. gauge lengths. Later, more accurate measurements were made along the median lines of the upper and lower edges of the bars, using 2-in. gauge lengths. Unless otherwise indicated, the calculations herein reported are based on strains measured by the later method. Column (10) of Table 8 shows that for oblique loading and values of P/A of 14 000 lb. per sq. in., or higher, the ratio,

$$\frac{\text{Average stress in lacing-bars calculated from strain}}{\text{Average stress in lacing-bars calculated from load}}$$

is 21 to 24% less than unity. It is of interest to note that the calculated maximum "load" stress, Column (8) of Table 8, at which the thin bars of Column No. 4 buckled, was practically the same as the average ultimate load for three similar lacing-bars tested singly as columns (Column (11) Table 8).*

TABLE 8.—AVERAGE UNIT STRESSES IN LACING-BARS OF COLUMNS NOS. 4, 5, 5*A*, AND 6, UNDER OBLIQUE LOADS.

(Strains measured on 28 to 32 bars in each column.)

Column No.	Nominal Size of Bars, in Inches.			$\frac{l}{r}$.	Load on column, in pounds.	Shear on column, in pounds.	Average Unit Stress in Lacing-Bars Calculated from		Ratio, $\frac{B}{A}$.	Average ultimate compressive strength of four bars, in pounds per square inch.
	Length.	Width.	Thickness.				Loads, in pounds per square inch, *A*.	Strains, in pounds per square inch, *B*.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
4	10¼	2¼	3/16	188	167 800	8 390	11 260	9 815	0.87	......
					202 200	10 110	13 550	11 920	0.88	......
					*262 200	13 110	17 600	†	....	17 200
5	10¼	2½	¼	141	*258 700	12 940	12 100	......	....	19 000
5*A*	10¼	2¼	¼	188	169 300	3 390	3 370	2 810	0.83	......
					169 300	8 465	8 410	7 300	0.87	......
					227 400	11 370	11 280	9 825	0.87	......
					*259 000	12 950	12 860	......	....	19 800
6	10¼	2½	⅜	95	164 500	8 225	4 930	3 745	0.76	......
					221 000	11 050	6 620	5 250	0.79	......
					*271 500	13 580	8 120	......	....	20 800

* Load at failure of column.
† Bars buckled.

Owing to imperfections in the connections and irregularities in the lacing-bars, the observed average strain for the lacing-bars varied considerably. If one considers variation among the tension bars, or among compression bars, the data in Table 9 indicate that the variation was least among the thin bars of Column No. 4; also, that the stress variation between bars became less as the loads increased. In the test of Column No. 4 the bars carrying compression were the outer bars on both sides of the column; in Columns Nos. 5*A* and 6, the bars carrying compression were the outer bars on one side and the inner bars on the other side of the columns. Comparing the tabulated information for Column No. 4 with that for the other columns, it appears from Column (7) of the lower part of Table 9 that a more equal distribution of stress between the inner and the outer bars was obtained for Columns Nos. 5*A* and 6 than for Column No. 4.

---

[5] See Equation (32), p. 1242.

TABLE 9.—VARIATION IN OBSERVED STRESSES IN LACING-BARS OF COLUMNS Nos. 4, 5*A*, AND 6 UNDER OBLIQUE LOADS.

(Strains measured on 28 to 33 bars in each column.)

Column No. (1)	Load on column, in pounds. (2)	Obliquity of load, percentage. (3)	UNIT STRESSES IN BARS FROM STRAINS. Kind. (4)	Minimum. (5)	Maximum. (6)	Average. (7)	Range from Average, Percentage. Below. (8)	Above. (9)
4	167 800	5	Tension	9 080	15 130	11 280	20	34
			Compression	6 040	10 700	8 350	28	28
	202 200	5	Tension	10 000	16 270	12 640	21	29
			Compression	8 610	13 480	11 200	23	20
	202 200	5	Both	8 610	16 270	11 920	28	36
5*A*	169 300	2	Tension	700	3 940	2 370	71	66
			Compression	1 160	5 100	3 250	64	57
	169 300	5	Tension	5 800	8 590	7 180	19	20
			Compression	5 340	9 060	7 420	28	22
	227 400	5	Tension	7 680	13 250	9 570	20	38
			Compression	8 820	12 090	10 080	12	20
	227 400	5	Both	7 680	13 250	9 825	22	35
6	164 500	5	Tension	3 250	8 150	4 790	32	70
			Compression	1 160	4 640	2 700	57	72
	221 000	5	Tension	3 710	8 850	5 400	31	64
			Compression	3 020	8 150	5 100	41	60
	221 000	5	Both	3 020	8 850	5 250	42	69
			Position of bar					
4	202 000	5	Inner	10 000	16 270	12 460	21	29
			Outer	8 610	13 480	11 200	23	20
5*A*	169 300	2	Inner	700	5 100	2 600	73	96
			Outer	1 160	3 940	3 020	62	30
	227 400	5	Inner	7 680	11 850	9 720	21	22
			Outer	7 690	13 250	9 930	23	33
6	221 000	5	Inner	3 020	7 690	5 130	41	50
			Outer	3 710	8 850	5 370	31	65

Table 10, Column (5), shows that for the high column loads the ratio of:

$$\frac{\text{Average ``strain'' stress in the most strained lacing-bar}}{\text{Average stress calculated from the load}}$$

in a certain tension bar of Column No. 6[6] reached the high value of 1.65. For compression bars, however, the ratios in Column (6), Table 10, are considerably less. The average "strain" stress in lacing-bars was also calculated for the lacing-bar panel, in which the two most highly strained bars were found. The ratio:

$$\frac{\text{Average ``strain'' stress in the lacing of the most strained panel}}{\text{Average ``load'' stress in the lacing}}$$

---

[6] It should be noted that the strains in the heavy bars of Column No. 6 were considerably smaller than those in the bars of the other columns, and the errors in measurement were proportionately greater.

shown in Column (7), Table 10, indicates that in the worst case one panel of Column No. 6^6 exhibited a ratio of 1.3; for the other columns, the ratio was less than 1.10.

TABLE 10.—COMPARISON OF STRESSES IN LACING-BARS CALCULATED FROM LOAD WITH STRESSES CALCULATED FROM STRAIN, COLUMNS NOS. 4, 5*A*, AND 6.

(Strains measured on 28 bars in each column.)

Column No.	Load on column, in pounds.	Obliquity of load, percentage.	Average unit stress in lacing-bars from Load (*A*), in pounds per square inch.	RATIO, $\frac{\text{STRAIN STRESS}}{\text{LOAD STRESS } (A)}$ FOR: Bar with Most Strain Considering: All bars.	Bar with Most Strain Considering: Compression bars only.	Panel with most strain.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
4	167 800	5	11 260	1.34	0.95	1.02
	202 200	5	13 550	1.20	0.99	1.00
5*A*	169 300	2	3 370	1.51	1.51	1.10
	169 300	5	8 410	1.08	1.08	0.94
	227 400	5	11 280	1.17	1.07	1.00
6	164 500	5	4 930	1.65	0.94	1.30
	221 000	5	6 620	1.34	1.23	1.28

10.—Tests of Individual Lacing-Bars.—Compression tests were made on twenty individual lacing-bars, 10¼ in. long, center to center, loaded in the manner indicated in Fig. 19. Measurements of crookedness at mid-length were made on a number of these bars and the results are recorded in Columns (6) and (7), Table 11. In all cases save one, the bars which were appreciably bowed outward, away from the channels, showed lower strength.

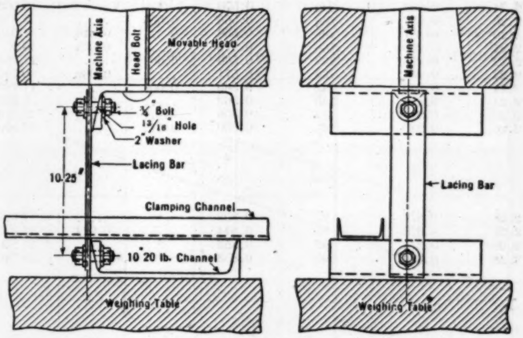


FIG. 19.—METHOD OF TESTING LACING-BARS.

To secure evidence concerning the magnitude of eccentricities in lacing, Bars 27*A*, 52*A*, and 61*A* were tested with Martens mirror extensometers attached to the middle 2 in. of each broad face. Also, in the column tests, measurements were made on the front faces as well as the top and bottom faces of fourteen bars in Column No. 4, and of ten bars each in Columns Nos. *5A* and 6. From data obtained in these two types of test, Fig. 20 has been prepared. Apparently, with the same diameter of bolt or rivet, the eccentricity is influenced markedly by the thickness of the bar. The eccentricities found in the bars tested separately under loads approximating 9 000 lb. per sq. in. were somewhat higher than the mean for the outer bars in the column tests. The lower eccentricities of the inner compression bars in the column tests are noteworthy. This evidence indicates that columns laced on two sides will have somewhat more uniform shearing strength and rigidity if the inner bars are sloped in opposite directions on the two sides.

TABLE 11.—RESULTS OF COMPRESSION TESTS ON LACING-BARS.

(Arrangement for testing shown in Fig. 19. Speed of movable head of machine was 0.06 in. per min. All specimens buckled outward at failure.)

Bar No. (1)	Dimensions, in Inches. Width. (2)	Dimensions, in Inches. Thickness. (3)	Slenderness ratio, $\left(\frac{l}{r}\right)$. (4)	Area, in square inches. (5)	Bowed at Middle, in Inches. In. (6)	Bowed at Middle, in Inches. Out. (7)	Maximum load, in pounds. (8)	Maximum unit load, in pounds per square inch. (9)
47*A*	2.24	0.189	188	0.423	*	.....	7 000	16 500
47*B*	2.28	0.189	188	0.431	.....	0.021	6 380	14 800
48*A*	2.27	0.187	190	0.425	*	.....	8 000	18 800
48*B*	2.26	0.186	191	0.421	*	.....	7 930	18 800
Average...	....	.....	....	.....	.....	.....	.....	17 200
51*A*	2.26	0.246	144	0.556	*	.....	12 000	21 600
51*B*	2.26	0.247	143	0.558	*	.....	10 360	18 600
52*A*	2.24	0.254	140	0.570	*	.....	10 560	18 500
52*B*	2.20	0.257	138	0.566	0.005	.....	9 810	17 300
53*A*	2.20	0.257	138	0.566	.....	0.014	11 040	19 500
53*B*	2.24	0.256	138	0.574	.....	0.023	10 550	18 400
54*A*	2.22	0.255	139	0.566	0.006	.....	11 730	20 700
54*B*	2.23	0.256	138	0.571	.....	0.039	10 790	18 900
55*A*	2.24	0.250	142	0.560	0.006	.....	11 780	21 000
55*B*	2.24	0.254	140	0.569	.....	0.040	9 950	17 500
56*A*	2.26	0.247	143	0.558	.....	0.024	10 370	18 600
56*B*	2.26	0.248	143	0.560	0.012	.....	13 550	24 200
Average...	....	.....	....	.....	.....	.....	.....	19 600
61*A*	2.28	0.374	95	0.853	*	.....	17 000	19 900
61*B*	2.26	0.373	95	0.844	.....	0.014	17 330	20 500
62*A*	2.28	0.374	95	0.853	*	.....	17 820	20 900
62*B*	2.28	0.374	95	0.853	*	.....	18 740	22 000
Average...	....	.....	....	.....	.....	.....	.....	20 800

* No measurements made.

The maximum unit loads from the individual compression tests of lacing-bars have been plotted against corresponding slenderness ratios in Fig. 21. It will be observed that the plotted points lie well above the line representing the Talbot and Moore equation, $P/A = 21\,400 - 45\,\frac{l}{r}$.[7] It is likely that the discrepancy is partly due to difference in size of fastening bolts. Bolts ½ in. in diameter were used by Talbot and Moore; in the Wisconsin tests they were ¾ in. in size.

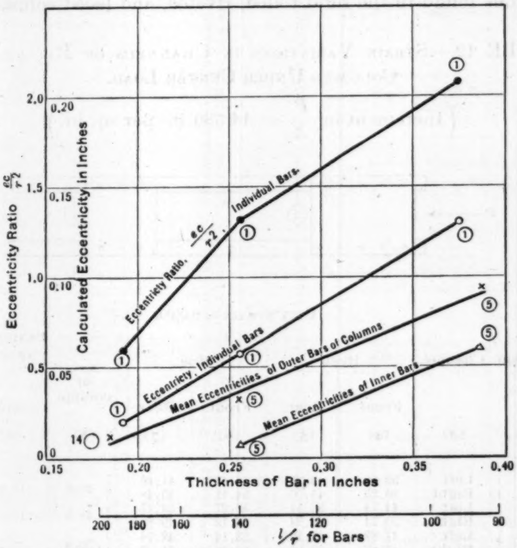


FIG. 20.—ECCENTRICITIES IN LACING-BARS UNDER COMPRESSION.

From the data now available, it appears that compression tests on individual lacing-bars yield safe results as a basis for design. More tests of this nature are much needed to supplement the meager fund of information.

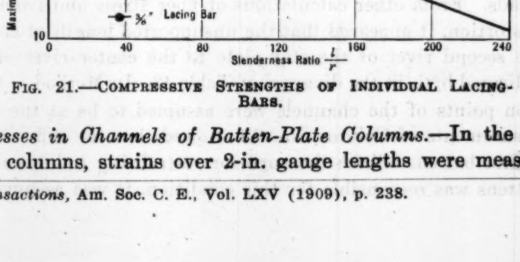


FIG. 21.—COMPRESSIVE STRENGTHS OF INDIVIDUAL LACING-BARS.

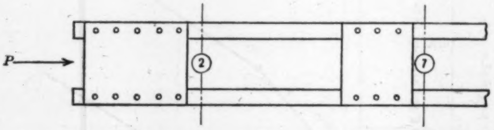
11.—Stresses in Channels of Batten-Plate Columns.—In the tests of the batten-plate columns, strains over 2-in. gauge lengths were measured on the outer fibers of the channel webs and toes at sections close to the lower stays and battens, and in Column No. 8*A*, in the middle of the unsupported channel lengths of the lower panel. The data from the tests made with central loading are assembled in Table 12. These results show that the fiber stress variations for this condition are not unduly large, but are comparable with variations found in the solid rolled, riveted, and laced columns.

---

[7] See *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 238.

TABLE 12.—STRAIN VARIATIONS IN CHANNELS OF BATTEN-PLATE COLUMNS UNDER CENTER LOAD.

$$\left(\text{Increment in } \frac{P}{A} = 14\,580 \text{ lb. per sq. in.}\right)$$



Column No.	Section.	Channel.	Unit Strain in 0.00001.					Ratio: Bending Strain / Axial Strain × 100.	
			Heel.		Toe.		Axis of column.		
			Front.	Rear.	Front.	Rear.		Average.	Maximum.*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
7	2	Left	52.50	43.75	56.43*	41.16	48.0	11.0	17.6
		Right	46.25	45.00	54.11	55.48			
	7	Left	44.74	52.11	48.27	34.47	48.6	10.8	29.9
		Right	50.74	46.34	68.12	49.88			
8	2	Left	47.87	55.12*	53.14	42.74	50.8	6.3	8.6
		Right	47.87	52.98	53.14	51.27			
	7	Left	46.13	50.00	54.90*	44.66	49.5	7.6	11.0
		Right	51.78	51.22	41.23	52.98			
9	2	Left	62.66*	39.67	59.14	48.51	53.0	10.2	18.3
		Right	57.86	51.30	54.34	51.30			
	7	Left	61.22	56.50	54.18	47.71	54.4	9.0	18.9
		Right	50.66	49.73	64.74*	49.78			

* Maximum stresses.

When the batten-plate columns were subjected to 5% shear loadings, the bending stresses in the unsupported lengths of channel became large. Table 13 shows some of the fiber stresses calculated from strains and from loads at the more important sections. In calculating the fiber stresses for the end panels, the results from two of the methods tried are presented. Each channel between plates was assumed to act as a fixed-end beam carrying axial end loads. From other calculations of fiber stress and from calculations for shear distortion, it appeared that the unsupported length of channel should be from the second rivet of the stay-plate to the center rivet of the batten-plate, as indicated by l_c in the diagram of Table 13. In Method (*a*), (Table 13), the inflection points of the channels were assumed to be at the middle of l_c. Strain measurements of Column No. 8*A* showed that the inflection point was nearer the first batten. Since the superior restraining influence of the stays over the battens was responsible for this condition, it was assumed in Method

TABLE 13.—COMPARISON OF CALCULATED AND MEASURED STRESSES IN CHANNEL FLANGES OF BATTEN-PLATE COLUMNS OF SERIES (*A*) UNDER 5 PER CENT. SHEAR.

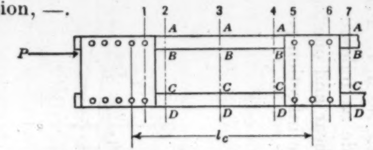
Column Data:

Length = 10 ft.

Material = two 10-in., 20-lb. channels.
four 12 by 13 by ¼-in. splice-plates.

Gauge length, 2 in.; Tension, +; Compression, —.

Column No. 7, two 9 by 12 by ¼-in. batten-plates.
" Nos. 8 and 8*A*, four 9 by 12 by ¼-in. batten-plates.
" No. 9, eight 9 by 12 by ¼-in. batten-plates.



Column No.	Section considered and assumption.*	Unsupported length of channel, in inches.	Applied load, *P*, in 1 000 lb.	Moment on one channel at section, in inch-pounds.	Unit Stresses on Extreme Fibers of Left Channel in 1 000 Pounds per Square Inch.				Unit Stresses on Extreme Fibers of Right Channel in 1 000 Pounds per Square Inch.			
					Heel *A*.		Toe *B*		Toe *C*.		Heel *D*.	
					Calculated.	Measured.	Calculated.	Measured.	Calculated.	Measured.	Calculated.	Measured.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
7	2*b*	49⅜	66.0	−0.401 *P*	−12.4	−13.2	+13.5	+15.7	−24.7	−26.0	+ 1.2	+ 1.3
8*A*	2*a*	32¾	96.7	−0.269 *P*	−16.9	−17.7	+ 9.4	+13.1	−26.5	−30.2	− 0.2	+ 1.9
	3*a*	32¾	96.7	−0.006 *P*	−11.3	−12.6	−10.7	− 7.5	− 6.4	−10.2	− 5.8	− 5.4
	4*a*	32¾	96.7	+0.256 *P*	− 5.7	− 4.2	−30.8	−29.1	−13.7	+10.2	−11.4	−12.1
	2*b*	32¾	96.7	−0.298 *P*	−17.4	−17.7	+11.8	+13.1	−28.9	−30.2	+ 0.3	+ 1.9
	3*b*	32¾	96.7	−0.036 *P*	−11.8	−12.6	− 8.3	− 7.5	− 8.8	−10.2	− 5.3	− 5.4
	4*b*	32¾	96.7	+0.227 *P*	− 6.3	− 4.2	−28.4	−29.1	+11.3	+10.2	−10.9	−12.1
	7*c*	33¼	96.7	−0.260 *P*	−14.1	−14.7	+11.4	+15.4	−28.5	−29.8	− 3.0	+ 0.2
	1*b*	32¾	96.7	−0.376 *P*	−19.3	......	−19.1	......	−34.8	......	+ 2.0	......
	5*b*	32¾	96.7	+0.305 *P*	− 4.6	......	−34.5	......	+21.6	......	−12.8	......
8	1*b*	32¾	117.5†	−0.376 *P*	−34.0	......	+31.3	......	−60.0	......	+ 8.6	......
9	2*b*	19⅛	137.6	−0.124 *P*	−19.4	−21.8	− 2.5	+ 2.7	−20.6	−23.2	− 3.8	− 3.1

* (*a*) Point of inflection at middle of unsupported length (l_c).
(*b*) Point of inflection 0.536 of unsupported length (l_c) from second rivet of stay-plate.
(*c*) Point of inflection midway between center rivets of adjacent battens.

† Maximum load.

(*b*), (Table 13), that the segments of the unsupported length of the top and bottom panels were directly proportional to the number of rivets per inch of adjoining plate. Hence, the length of segment adjacent to the stay-plates $= \frac{\frac{5}{13}}{\frac{5}{13} + \frac{3}{9}} \, l_c = 0.536 \, l_c$. If one compares the figures in bold-faced type, Table 13, it is evident that stresses calculated by Methods (*b*) and (*c*) check closely the strain stresses for these highly stressed fibers. To investigate points of greatest fiber stress, calculations at maximum loads by Method (*b*) were made for Sections 1 and 5 through rivets, also by Method (*c*) for Section 6. At the flange toes (*B*) of Section 1, the calculated fiber stress for Column No. 8 is 31 300 lb. per sq. in. tension and at Toes *C* of the same section, 60 000 lb. per sq. in. compression. Under the latter stress, excessive lateral bending occurred which threw more tensile stress on Toe *B*. Finally, the metal necked down and broke in tension. Similar failures occurred in Columns Nos. 7 and 9, although Column No. 9 carried a load nearly as large as the laced-channel columns.

The detailed calculation for stresses at Points *A, B, C,* and *D* in Section 2 of Column No. 8*A* by Method (*b*) were made as follows: Each channel was assumed to carry one-half the transverse shear and one-half the bending moment (Fig. 22). Then, the distance from the inflection point to Section 2 is (32.75 × 0.536) — 5.625 = 11.92 in.

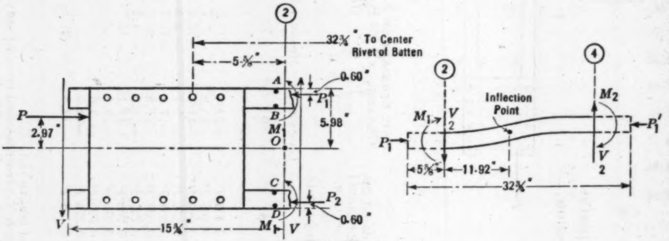


FIG. 22.

Then,

$$M_1 = 11.92 \, \frac{V}{2} = 0.298 \, P$$

$$\Sigma M_0 = 2.97 \, P - (0.05 \, P \times 15.75) - 5.38 \, (P_1 - P_2) - 0.596 \, P = 0$$

Since $P = P_1 + P_2$, $P_1 = 0.647 \, P$ and $P_2 = 0.353 \, P$.

Using $p = \frac{P}{A} + \frac{M c}{I}$ to find the unit fiber stress, p, $A = 5.65$ sq. in.; for the heel, $\frac{c}{I} = 0.220$; and for the toe, $\frac{c}{I} = 0.792$. Hence,

$$p_A = -\frac{0.647 \, P}{5.65} - (0.298 \, P \times 0.22) = -0.180 \, P$$

$$p_B = -\frac{0.647 \, P}{5.65} + (0.298 \, P \times 0.792) = +0.122 \, P$$

$$p_C = -\frac{0.353\,P}{5.65} - (0.298\,P \times 0.792) = -\,0.299\,P$$

$$p_D = -\frac{0.353\,P}{5.65} - (0.298\,P \times 0.22) = +\,0.003\,P$$

Substituting $P = 96\,700$ lb., the stresses given in Table 13 were obtained.

12.—Shear Stress in Batten-Plates.—Strain measurements were made on the stay and batten-plates of Columns Nos. 7 to 9; also on the stay-plates of Columns Nos. 4, *5B*, and 6, as indicated in Figs. 4 to 7. It was found, however, that the strains on the outer faces of these plates were much larger than the average for the considered gauge line. Hence, in order to secure average strains, measurements on both sides of every plate had to be made. Although difficult, such measurements were made with the 2-in. Olsen gauge on the lower battens and stays of Columns Nos. 8 and *8A*.

Plate measurements parallel and transverse to the axis of the column showed that these strains were insignificant. The strains on the 45° diagonals in these plates, however, were appreciable. From strains the shear stress in the plates was calculated, assuming Poisson's ratio to be 0.25, by,

$$S_S = \frac{4}{15}\,(4\,\varepsilon_1 + \varepsilon_2)\,E \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

In this expression, ε_1 is the unit strain in one diagonal, and ε_2, that in the diagonal normal to it. The shear stress in the plates was also calculated by the horizontal shear formula for rectangular sections. The stress in each of the two plates will be:

$$S_S = \frac{3}{4}\,\frac{V}{b\,d} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

In Equation (6), V is the stress difference on either side of a batten in one channel, b, the thickness of the plate, and, d, the length of the plate measured along the column. The results of both calculations are given in Table 14. When one considers the small magnitude of the strains measured, the agreement between the stresses calculated from strain and those calculated from shear is very satisfactory. Hence, the designer may safely use the horizontal shear formula (Equation (6)) to determine the shearing stresses in the battens of such columns.

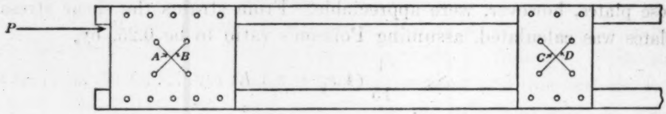
TESTS OF SERIES (*B*): ECCENTRIC LOADS

13.—Method of Testing.—These tests were made on solid rolled **H**-columns having values of $\frac{l}{r}$ from 20 to 158. The purpose of the tests was, in general, to compare the behavior of such columns furnished under ordinary commercial conditions, with that of the ideal column; that is, to ascertain the effect of crookedness, eccentricity of rolling, and other variations, on the strength. The tests also furnished some valuable information on the effect of form on the strength of columns.

All columns were first centered in the machine with reference to the calculated gravity axis of the end sections, taking also into account any measured crookedness of the columns. Shims were used to secure even bearings in a direction parallel to the axis of the rollers of the bearing blocks. Further correction was made in the setting by applying loads up to about 10 000 lb. per sq. in., observing deflection at the center, and taking strain-gauge readings over the center section. After this careful centering, a center loading test up to 15 000 lb. per sq. in., or when possible up to the Euler load, was run. The columns were then shifted a calculated amount to give the desired eccentricity. Fig. 27 shows Column No. *H*7 ready to be tested.

TABLE 14.—COMPARISON OF CALCULATED AND MEASURED SHEAR STRESSES IN BATTEN-PLATES OF COLUMNS NOS. 8 AND 8*A* UNDER 5 PER CENT. SHEAR LOADS.

(Primed letters refer to gauge lines paralleling those shown on the opposite side of column. Nominal dimensions of stay-plates, 13 by 12 by $\frac{1}{2}$ in.; batten-plates, 9 by 12 by $\frac{1}{4}$ in.)



Column No.	Batten or stay and gauge line considered.	Load applied to column, in pounds.	Shear per batten, V, in pounds.	Measured unit strain on 45° 2-in. gauge line, $10^{-6}\epsilon$*.	Unit Shear Stress on Battens, in Pounds per Square Inch. Calculated, $S_s = \frac{3}{2}\frac{V}{A}$.	Measured, $S_s = \frac{4}{15}(4\epsilon_1 + \epsilon_2)E$.	Ratio, calculated to measured.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
8	Lower *A*	113 400	8 330	−24.85	5 460	5 270	0.88
	Batten *B*			+31.25		7 750	
	" *A′*			−25.65		5 960	
	" *B′*			+25.65		5 960	
	Mean			26.85		6 225	
8*A*	Lower *A*	96 700	7 100	−9.6	2 820	2 100	1.10
	Stay *B*			+11.2		2 720	
	" *A′*			−11.2		2 540	
	" *B′*			+12.0		2 850	
	Mean			11.0		2 550	
8*A*	Lower *A*	96 700	6 240	−19.2	4 620	4 270	0.96
	Batten *B*			+21.6		5 200	
	" *A′*			−19.2		4 140	
	" *B′*			+23.2		5 690	
	Mean			20.8		4 825	

* Elongation, +; compression, −.

Columns Nos. *H*2, *H*3, *H*4, and *H*5 were first loaded within the elastic limit with the eccentricity perpendicular to the web, then turned and tested to failure with the eccentricity in the plane of the web. Columns Nos. *H*1, *H*6, *H*7, *H*8, *H*9, and *H*10 were first loaded within the elastic limit with the eccentricity in the plane of the web, then turned and tested to failure with the eccentricity perpendicular to the web. Column No. *H*1 (the shortest) was then straightened and tested to failure with the eccentricity in the plane of the web. Thus, all the columns were tested within the elastic limit with an eccentricity in each direction, four of the columns to failure by bending perpendicular to the web, six to failure by bending parallel to the web, and one (Column No. $H1$) also to failure by bending perpendicular to the web. To supplement the data for the shortest length tested, Column No. $H10B$ was cut from the lower end of Column No. $H10$ and tested to failure with an eccentricity in the plane of the web.

Eccentricities were chosen so as to make the value of $\frac{e\,c}{r^2}$ (eccentric ratio) equal to 0.5, 1.0, or 2.0; in this expression, e is eccentricity; c, distance to the extreme fiber; and r, the radius of gyration. In every case failure tests were made with $\frac{e\,c}{r^2} = 1.00$. With this value, the fiber stress on the concave side of the column (neglecting the effect of deflection) would be twice the average, and on the convex side, zero. Deflections at the center were measured, also strains near the ends, quarter-points, and center, at the four corners (edges of flanges), center of each flange, and each side of the center of the web.

14.—Fiber Stresses at Maximum Loads.—A comparison of the calculated maximum unit fiber stresses with the yield point of the metal in these H-columns is given in Table 15. The calculated fiber stresses were computed by the formula:

$$p = \frac{P}{A}\left[1 + \frac{(e + D)\,c}{r^2}\right]$$

in which, D is measured deflection and e, eccentricity, including the effect of crookedness. The calculations assumed proportionality of stress and strain across the section and for values beyond the yield point give fictitious rather than real stresses. The results, however, serve for comparative purposes. The following significant results are noted:

(1) The maximum fiber stress is in every case considerably beyond the proportional limit of the material, but at "first scaling" the stress is considerably below the limit.

(2) For a column placed with its web perpendicular to the axis of the rollers (bending parallel to web), the maximum fiber stress averages somewhat greater than the yield point of the material, but is slightly less in two cases.

(3) For a column placed with its web parallel to the axis of the rollers (bending perpendicular to web), the maximum fiber stress is greater (generally very much greater) than the yield point of the material, and, therefore, greater than the maximum stress when tested in the other direction. The excess over the latter amounts to the following percentages: 41, 62, 42, 36, and 21, the last being for the longest column. These large differences are significant, and indicate a very considerable difference in the behavior of columns when the bending is in a plane parallel to an outstanding flange and when perpendicular to such a flange. This may be of importance when considering the application of formulas which take account of the form of section in terms of the ratio of c to r. Of course, the effect of the difference in the ultimate fiber stress here shown does not indicate a correspondingly large difference in the ultimate strength of the column. This will be discussed later.

TABLE 15.—COMPARISON OF CALCULATED MAXIMUM UNIT STRESSES IN H-COLUMNS FOR LOADS AT OR NEAR ULTIMATE WITH YIELD POINT OF MATERIAL.

(With rollers normal to web, $e = 2.93$ in., $r = 3.40$ in.; with rollers parallel to web, $e = 0.98$ in., $r = 1.98$ in.)

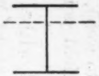
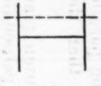
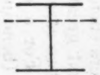
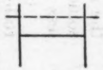
Column No.	Length, l, in inches.	$\frac{l}{r}$.	Direction of roller axes.	ULTIMATE LOAD, $\frac{P}{A}$, IN POUNDS PER SQUARE INCH.		DEFLECTIONS, D, IN INCHES, AT:		CALCULATED UNIT STRESSES, IN POUNDS PER SQUARE INCH, AT:		PROPERTIES OF MATERIAL, IN POUNDS PER SQUARE INCH.		
				At first scaling.	At ultimate.	First scaling.	Ultimate.	First scaling.	Ultimate.	Proportional limit in tension.	Average yield point of tension specimens.	Average ultimate, compression for whole prisms.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
H 1	39.1	11		17 000	20 700	....	....	33 900	41 400	33 600	37 400	38 800
	39.1	20		18 000	26 900	....	....	35 900	53 600	36 000	39 000	38 800
*H*10 *B*	39.6	12		18 200	21 000	0.02	0.18	26 400	43 100	34 800	37 400	37 600
H 6	39.6	20		16 000	28 850	0.03	0.25	32 700	65 800	39 700	43 000	38 800

TABLE 15.—(*Continued.*)

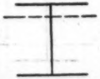
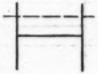
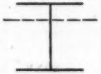
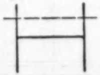
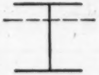
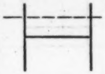
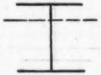
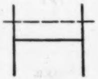
Column No.	Length, l, in inches.	$\frac{l}{r}$.	Direction of roller axes.	ULTIMATE LOAD, $\frac{P}{A}$, IN POUNDS PER SQUARE INCH. At first scaling.	ULTIMATE LOAD, $\frac{P}{A}$, IN POUNDS PER SQUARE INCH. At ultimate.	DEFLECTIONS, D, IN INCHES, AT: First scaling.	DEFLECTIONS, D, IN INCHES, AT: Ultimate.	CALCULATED UNIT STRESSES, IN POUNDS PER SQUARE INCH, AT: First scaling.	CALCULATED UNIT STRESSES, IN POUNDS PER SQUARE INCH, AT: Ultimate.	PROPERTIES OF MATERIAL, IN POUNDS PER SQUARE INCH. Proportional limit in tension.	PROPERTIES OF MATERIAL, IN POUNDS PER SQUARE INCH. Average yield point of tension specimens.	PROPERTIES OF MATERIAL, IN POUNDS PER SQUARE INCH. Average ultimate, compression for whole prisms.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
H 2	99.0	29		16 000	19 950	0.17	0.41	32 900	42 600	33 600	37 400	38 800
H 7	99.0	51		12 000	21 660	0.14	1.12	25 900	69 000	39 700	43 000	38 800
H 3	168.3	50		12 000	17 950	0.38	1.11	25 400	42 500	33 600	37 400	38 800
H 8	168.3	86		14 000	16 200	0.64	1.60	37 900	60 200	39 700	43 000	38 800
H 4	237.8	70		12 000	15 100	0.88	1.48	27 600	37 800	33 600	38 000	40 900

TABLE 15.—(*Continued.*)

Column No.	Length, l, in inches.	$\frac{l}{r}$.	Direction of roller axes.	ULTIMATE LOAD, $\frac{P}{A}$, IN POUNDS PER SQUARE INCH.		DEFLECTIONS, D, IN INCHES, AT:		CALCULATED UNIT STRESSES, IN POUNDS PER SQUARE INCH, AT:		PROPERTIES OF MATERIAL, IN POUNDS PER SQUARE INCH.		
				At first scaling.	At ultimate.	First scaling.	Ultimate.	First scaling.	Ultimate.	Propor-tional limit in tension.	Average yield point of tension specimens.	Average ultimate, compression for whole prisms.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
H 9	237.5	121		10 200	10 620	1.30	2.80*	33 900	51 600	42 600	45 300	39 400
H 5	306.9	90		11 500	12 600	1.71	2.55	29 700	36 200	33 100	36 400	35 300
*H*10	306.9	158		7 750	7 750	2.85	3.48	38 800	44 000	34 800	41 800	37 600

* Estimated.

Table 16 gives some of the stresses from strain measurements as compared to calculated stresses under loads within the elastic limit. Unit stresses were calculated from strains as follows: For Fiber A,

$$p_A = \frac{P}{A}\left(\frac{4\,e_A}{e_A + e_B + e_C + e_D}\right)$$

The 8-in gauges near the top and bottom of the columns were centered 8 in. from the ends. (See Fig. 8.)

Considering all **H**-columns and unit loads, *P/A,* sufficient to produce stresses of 16 000 to 27 000 lb. per sq. in. on the outer concave fibers at mid-section, the average variation of the average gauge stress was only 2.6% from the calculated stress, and the maximum variation was 11.7 per cent.

A variation between individual and average gauge readings (large only at end sections) indicated the effect of irregularity of end condition in a plane parallel to the axis of the roller (fixed direction), and shows the difficulty of flat end tests.

15.—Measured and Calculated Deflections.—Figs. 23, 24, and 25 show comparisons between observed and calculated central deflections, the calculated deflections being determined by the theoretical formula for eccentric load, as follows:

$$D = e\left(\sec\sqrt{\frac{P}{E\,I}}\,\frac{l}{2\,r} - 1\right)$$

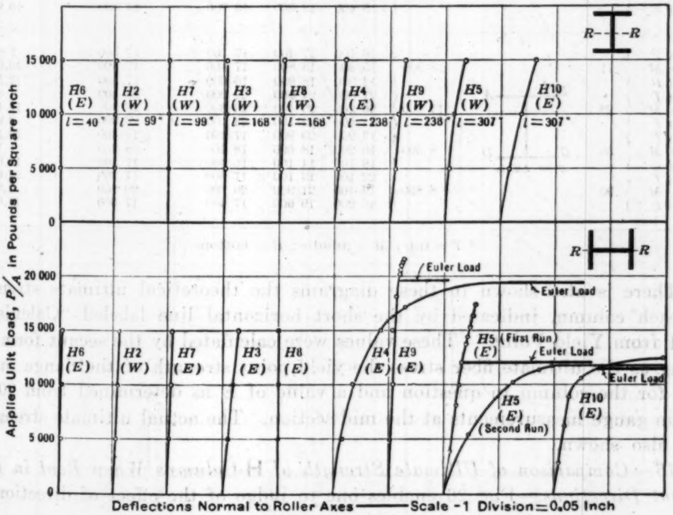


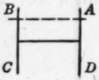
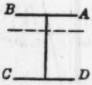
FIG. 23.—LOAD-DEFLECTION CURVES FOR 8-INCH. 32-POUND, H-COLUMNS, UNDER CENTER LOADINGS.

The value of E used was that obtained from strain measurements on the center section, and r was the measured value. Note the very close agreement

between observed and calculated deflections. Fig. 25 also shows the marked effect of the magnitude of the eccentricity of the load on the deflection of a long column like Column No. *H*4.

TABLE 16.—COMPARISON OF MAXIMUM FIBER STRESSES IN **H**-COLUMNS CALCULATED FROM STRAINS WITH STRESSES FROM LOAD AND DEFLECTION.

$$\left(\text{Eccentric loading, } \frac{ec}{r^2} = 1.\right)$$

Column No. and Section.* (1)	$\frac{l}{r}$. (2)	Position of roller axes. (3)	Load, $\frac{P}{A}$, in pounds per square inch. (4)	Unit Stresses in Outer Fibers on Concave Side of Column, in Pounds per Square Inch. Gauge *A*. (5)	Gauge *B*. (6)	Average Gauges *A* and *B*. (7)	Calculated, $S = \frac{P}{A}\left[1+\frac{(e+D)c}{r^2}\right]$. (8)	Percentage variation of gauge stress from average. (9)
H 1 *T*	20		7 900	16 500	14 700	15 600	15 800	5.8
M				16 300	15 300	15 800	15 700	3.2
B				16 000	15 300	15 700	15 700	1.9
H 2 *T*	51		8 950	18 000	16 300	17 200	17 400	4.7
M		*e* = 0.98 in.		19 400	20 600	20 000	19 700	3.0
B				16 700	18 400	17 600	17 600	4.5
H 3 *T*	87		8 900	16 600	20 000	18 300	18 300	9.3
M				20 400	20 400	20 400	21 700	0.0
B				19 700	15 500	17 600	18 200	11.9
H 10 *T*	159		5 910	13 600	13 600	13 600	12 800	0.0
M				19 600	18 900	19 300	19 800	1.6
B				15 400	12 300	13 900	13 300	10.8
H 1 *T*	11		8 900	16 700	17 400	17 100	17 500	1.8
M				15 300	18 800	17 100	17 500	10.0
B				14 000	18 000	16 000	17 500	12.5
H 2 *T*	29		11 900	21 800	24 000	22 900	23 400	4.8
M		*e* = 2.93 in.		26 200	20 400	23 300	23 900	12.4
B				24 000	22 100	23 100	23 300	3.9
H 3 *T*	50		8 900	13 900	20 900	17 400	17 400	20.1
M				18 200	18 600	18 400	18 200	1.1
B				18 700	14 100	16 400	17 500	14.0
H 10 *T*	90		8 890	22 100	13 100	17 600	17 300	25.6
M				19 400	21 900	20 700	20 800	5.8
B				15 200	19 600	17 400	17 600	12.6

* *T* = top ; *M* = middle ; *B* = bottom.

There is also shown in these diagrams the theoretical ultimate strength of each column, indicated by the short horizontal line labeled "Calculated *P/A* from Yield Point." These values were calculated by the secant formula, using as the ultimate fiber stress the yield point strength of the flange material for the column in question and a value of *E* as determined from 20-in. strain-gauge measurements at the mid-section. The actual ultimate strengths are also shown.

***16.—Comparison of Ultimate Strength of* H*-Columns When Bent in Different Directions.*—Fig. 26 enables one to judge of the effect of direction of bending on the ultimate strength for equal values of $\frac{l}{r}$ and on the actual, as compared to the theoretical, strength. The full line represents the theoretical strength for a fiber stress equal to the average tensile yield point of the**

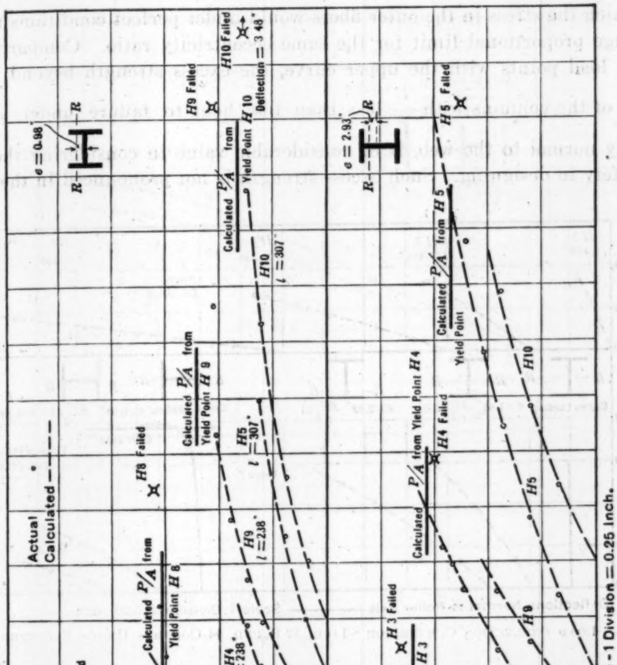


FIG. 24.—LOAD-DEFLECTION CURVES FOR 8-INCH, 32-POUND, H COLUMNS UNDER ECCENTRIC LOADINGS, $\frac{ec}{r^2} = 1$.

material when $\frac{e\,c}{r^2} = 1$. The dashed line shows the theoretical unit load (P/A) at which the stress in the outer fibers would under perfect conditions reach the average proportional limit for the same eccentricity ratio. Comparing maximum load points with the upper curve, the excess strength beyond the yield point of the columns with $\frac{l}{r}$ less than 100, bent to failure under an eccentricity normal to the web, is of considerable value in considering the margin of safety in designing. Such excess strength is not pronounced in the columns bent to failure under an eccentricity in the plane of the web. It should be noted that all these tests were made with the same $\frac{e\,c}{r^2}$, but that the value of e in the plane of the web was three times the value of e for the other position. Hence, the results do not show that a given **H**-column will be as strong as, or stronger against buckling normal to the web than against buckling parallel thereto, but only that the calculated extreme fiber stress at failure is likely to be higher in the first case than in the second.

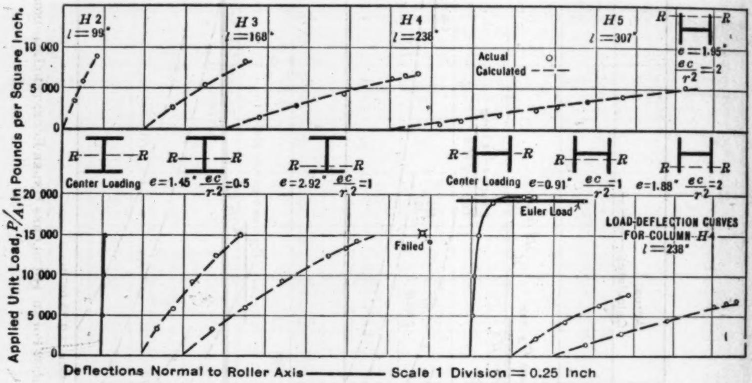


FIG 25.—LOAD DEFLECTION CURVES FOR 8-INCH, 32-POUND, H-COLUMNS UNDER ECCENTRIC LOADINGS.

Comparing the plotted points representing the first observed over-strain at the quarter-points or mid-section with the dashed line, Fig. 26, one perceives that the variability of strain distribution produced over-strain in many of the shorter columns at loads somewhat less than those calculated. This condition was more common for bending in the plane of the web. The lower part of Fig. 26 affords the assurance that for short **H**-columns bent in either of the aforementioned ways there is a comfortable margin of safety between the first appearance of strain lines and failure.

These **H**-column tests furnish satisfactory confirmation of the reliability and accuracy of the secant formula when the conditions are known. There is need, however, of further tests of this character on sections of different form, to assist in answering the question of the effect of form in column design.

The arrangement of apparatus for the tests of Series (*B*) is shown in Fig. 27.

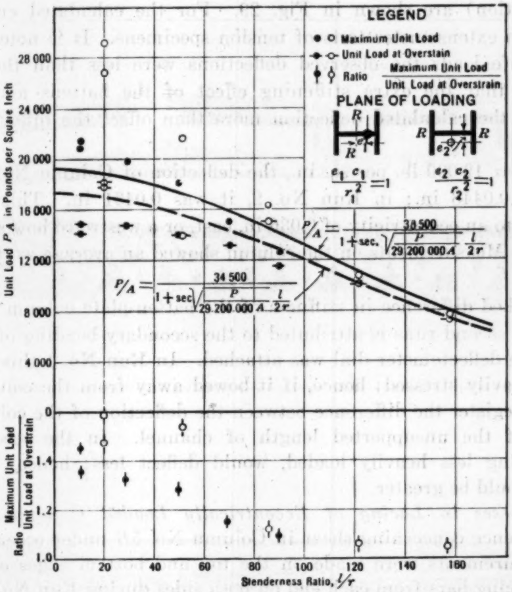


FIG. 26.

TESTS OF SERIES (*C*)

17.—Procedure in Testing.—In this series a laced channel column, No. 5*B*, and a batten-plate column, No. 8*A* (Fig. 28) were tested both under eccentric loads and under oblique loads. The purpose of the eccentric load tests was to ascertain if there was sufficient shear strain present to increase materially the lateral deflection of these columns. The function of the oblique loading tests was to supplement the data on shear distortion and stresses in battens and channels of Series (*B*) with results as already presented.

In the eccentric load tests, each column was first tested with $\frac{e\,c}{r^2} = 0.4$ and bent normal to the web planes without over-strain. It was then shifted to a parallel position with an eccentricity equal in amount but opposite in sign. Both center deflections and strains over 8-in. gauge lengths on the corner fibers near the ends and at the center were read during these tests. Following the oblique loading tests, Column No. 5*B* was tested to destruction under eccentric loads with $\frac{e\,c}{r^2} = 0.6$, *e* being normal to the webs; and Column No. 8*A* was crushed under eccentric loads with $\frac{e\,c}{r^2} = 1.0$, *e* being parallel to the webs.

18.—Deflection Data.—The load-deflection curves for these eccentric load tests and the deflection curves calculated by the secant formula (which neglects shear distortion) are shown in Fig. 29. For the calculated curves, E was derived from extensometer tests of tension specimens. It is noted that in all cases for safe loads the observed deflections were less than the calculated. This shows that the extra stiffening effect of the battens and lacing not included in the calculated deflection more than offset the influence of shear distortions.

At $P/A = 12\,000$ lb. per sq. in., the deflection of Column No. 5*B*, in Run No. 1, was 0.0448 in.; in Run No. 2, it was 0.0431 in. This discrepancy corresponds to an eccentricity of 0.036 in. east, or a westward bow at mid-length of 0.036 in. Measurements on the column showed an average westward bow of 0.023 in.

The marked difference in stiffness of the batten-plate column (No. 8*A*) in the first and second runs is attributed to the secondary bending of the channel to which the deflectometer dial was attached. In Run No. 1 this channel was the more heavily stressed; hence, if it bowed away from the column axis the dial would register the difference between the deflection of the column and the deflection of the unsupported length of channel. In the second run the channel, being less heavily loaded, would deflect less; hence, the recorded deflection would be greater.

19.—Stresses in Lacing of Eccentrically Loaded Columns.—To secure further evidence concerning shear in Column No. 5*B* under eccentric loading, strain measurements were made on the top and bottom edges of the second and third lacing-bars from each end on both sides during Run No. 6 to failure. From these strains the calculated average stress in the bars, at $P/A = 18\,000$ lb. per sq. in., was 1 140 lb. per sq. in. At this load stresses in the eight bars varied from 230 to 2 300 lb. per sq. in. These data, like the deflection-test evidence, indicate that the transverse shear in the eccentrically loaded column is very small. The maximum stress in the lacing-bars tested indicates a shear of $V = 0.011P$.

These general results as regards shear might have been predicted, as the theoretical shearing stress in a column with such a low slenderness ratio $\left(\frac{l}{r} = 22\right)$ is very small. In this case for $P/A = 18\,000$ lb. per sq. in., the theoretical end shear is only $0.0037P$, an amount too small to change the deflection sensibly. For more slender columns the effect would be greater. This question is discussed further in Part II.

20.—Maximum Fiber Stresses at Failure.—The failure of these columns was instructive. In the test of Column No. 5*B*, scaling was observed on the concave outer fibers just below the upper battens at $P/A = 16\,000$ lb. per sq. in., and failure occurred at $P/A = 19\,000$ lb. per sq. in., by bending normal to the rollers. Column No. 8*A* deflected normal to the rollers as a unit until the outer concave fibers were stressed well beyond the yield point; then the middle unsupported portions of channel suddenly buckled laterally as shown in Fig. 28. Scaling was not observed prior to the imposition of the maximum load.

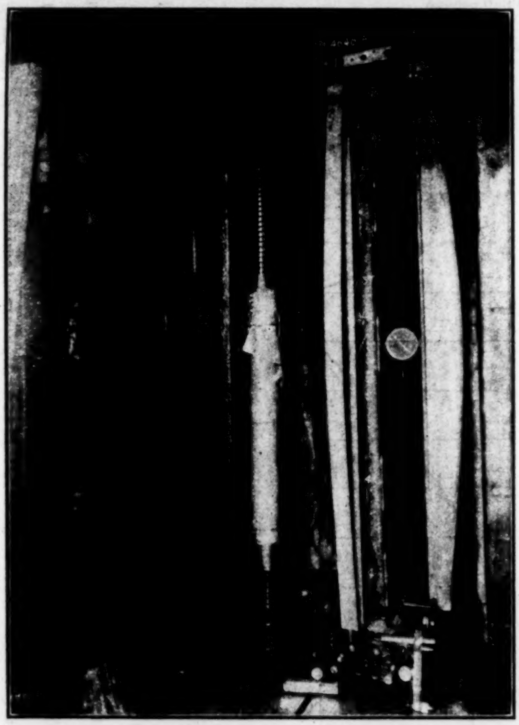


FIG. 27.—COLUMN NO. *H*7 READY FOR AN ECCENTRIC LOAD TEST.

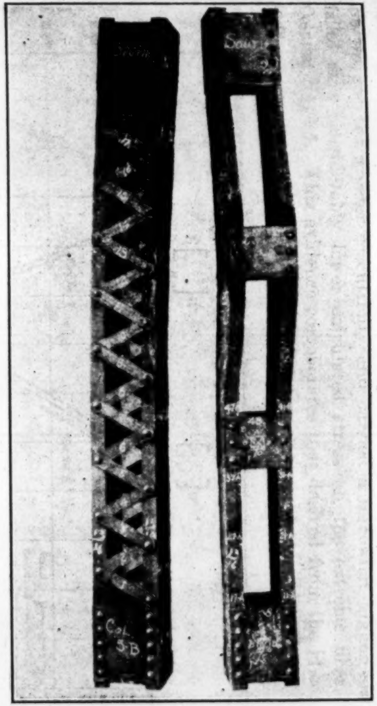


FIG. 28.—FAILURES OF COLUMNS NOS. 5*B* AND 8*A* TESTED UNDER ECCENTRIC LOADS.

Additional evidence of the influence of the form of cross-section on the maximum calculated fiber stress is afforded by the results of these two tests, which are summarized in Table 17. It will be seen that, although Column No. 5*B* had the greater eccentricity and carried a somewhat higher load, it failed at a considerably lower calculated stress on the extreme fiber than Column No. 8*A*. This evidence corroborates that derived from the **H**-column tests.

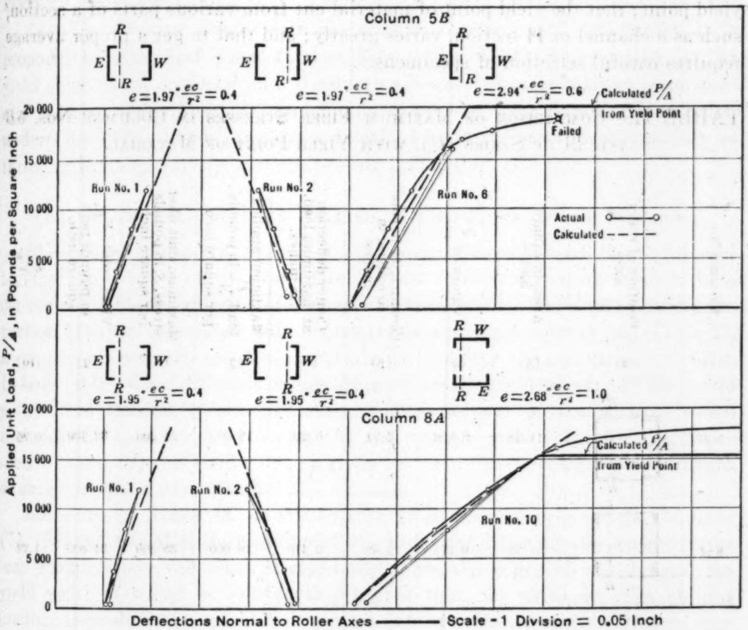


FIG. 29.—LOAD-DEFLECTION CURVES FOR COLUMNS NOS. 5*B* AND 8*A* OF SERIES (*C*).

PART II.—GENERAL REPORT

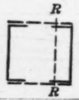
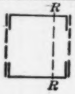
21.—The Report of 1926.—In the report of 1926,[8] the subject was divided into the following subdivisions:

(A) Strength as Related to Yield Point.—This topic was discussed with reference to results of tests on columns with slenderness ratios, $\frac{l}{r}$, not greater than 100 and with flat ends or flat-end behaviors. It was shown that for $\frac{l}{r} = 50$, the average ultimate strength was about 95% of the tensile yield point of the material composing the column, ranging in quality from nickel steel with a yield point of 56 000 lb. per sq. in. to low carbon steel with a yield point of 25 000 lb. per sq. in. For $\frac{l}{r} = 100$, the average strength was about 88% of the yield point. The great majority of the results was larger than would be calculated by assuming an eccentric ratio, $\frac{e\,c}{r^2} = 0.1$, and by taking due account of the slenderness ratio, $\frac{l}{r}$. It was also shown that the tensile yield point of steel of various grades is closely equal to the compressive yield point; that the yield point of material cut from various parts of a section, such as a channel or **H**-section, varies greatly; and that to get a proper average requires careful selection of specimens.

---

[8] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1485.

TABLE 17.—COMPARISON OF MAXIMUM FIBER STRESSES IN COLUMNS NOS. 5*B* AND 8*A* OF SERIES (*C*), WITH YIELD POINT OF MATERIAL.

Column No. (1)	Position of roller axes. (2)	Area, in square inches. (3)	Values of $\frac{c}{r^2}$. (4)	Eccentricity normal to rollers, in inches. (5)	Center deflection at maximum load, in inches. (6)	Maximum applied load, $\frac{P}{A}$, in pounds per square inch. (7)	Maximum unit fiber stress, S, in pounds per square inch. (8)	Yield point, Y_F, in pounds per square inch. (9)	$\frac{S}{Y_F}$. (10)
5 *B*		11.38	0.203	2.91	0.210	19 000	31 200	32 400	0.96
8 *A*		11.30	0.373	2.68	0.410	18 000	38 800	31 800	1.22

(B) Results of Tests on Columns of All Lengths Showing Integral Action.—All available tests were plotted and discussed. It was shown that tests on flat-end columns and those pin-end tests that showed flat-end behavior gave results fairly represented by a parabolic curve beginning at the yield point for $\frac{l}{r} = 0$, and tangent to the Euler curve, $P/A = \frac{4\,\pi^2 E}{\left(\frac{1.12\,l}{r}\right)^2}$; and that tests on pin-end columns gave results fairly represented by a parabola beginning at the yield point and tangent to the Euler curve, $P/A = \frac{\pi^2 E}{\left(\frac{0.78\,l}{r}\right)^2}$.

(C) Effect of Various Details on Strength of Columns.—Under this head was discussed the data relative to eccentricities, it being shown that few tests gave values greater than those represented by the eccentric ratio, $\frac{e\,c}{r^2} = 0.1$, corresponding to a bending stress for short lengths of 10% of the direct stress. The unsupported lengths of column components were discussed as regards thickness of outside plates, and results were given to indicate that a ratio of rivet pitch to thickness equal to 20 was sufficiently low to develop the full strength of the column, and that current practice was conservative in this respect.

(D) Column Theories; The Column in a Truss; Working Formulas.—This proposed subdivision was left for a subsequent report.

22.—The Present Report.—In the general discussion of this report, it is proposed to give first some further data under Part (A), relating to the yield point of the material and to column strength; then to take up some general theories under Part (D) and finally to present some data and discussion under Part (C). The column tests described in Part I are of value mainly as throwing light on various matters coming under Part (C).

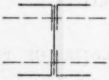
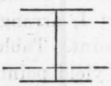
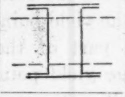
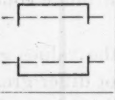
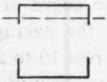
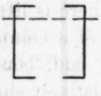
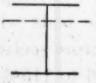
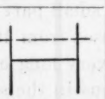
(A) COLUMN STRENGTH AS RELATED TO PROPERTIES OF THE MATERIAL

23.—Column Strength and Yield Point.—In Tables 18 and 19 are brought together the results of the tests described in Part I, arranged to show in a convenient form the relation of strength to yield point. Table 18 gives a comparison between calculated unit fiber stresses and yield points; and Table 19, a comparison between actual and calculated loads for the eccentric load tests. In both tables, three different values are used for the yield point: (1) The yield point from tension specimens taken from that part of the section which includes the most strained fibers; (2) the average yield point for the entire section; and (3) the yield point as given by the ultimate compressive strength of short prisms of the columns.

Referring to Table 18, it will be seen from the values given in Columns (7), (8), and (9), that the yield point values do not differ greatly except in the last group, where the most strained fibers are the edges of the flanges, the yield point of which is considerably higher than the average. The oblique loading tests showed a calculated maximum fiber stress 10 to 20% greater than the yield point. In these tests the fiber stress is a maximum at the end of the column, and diminishes toward the center so that there is little or no buckling action. The behavior is similar in nature to that of a column in a structure with heavy secondary bending moments at each end, bending the column into an **S**-shaped curve. All these columns were relatively short.

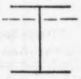
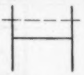
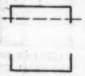
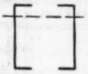
The eccentric load tests show two general results corresponding to the two conditions discussed in Part I, Section 16. Those tests where the deflection was in a direction such that a large part of the cross-section was stressed to the maximum, or nearly so, showed stresses from 96 to 115% of the yield point (Column (10), Table 18); those tests where but a small part of the section was stressed to the maximum, showed calculated stresses from 105 to 160% of the yield point, the small value of 105% being for a very long column. As stated in Part I, these excessive fiber stresses are fictitious in the sense that they are not actual stresses, but calculated values based on the elastic behavior of the material up to failure.

TABLE 18.—COMPARISON OF MAXIMUM UNIT FIBER STRESSES CALCULATED FROM MAXIMUM LOADS AND DEFLECTIONS WITH YIELD POINTS OF MATERIAL, WISCONSIN TESTS.

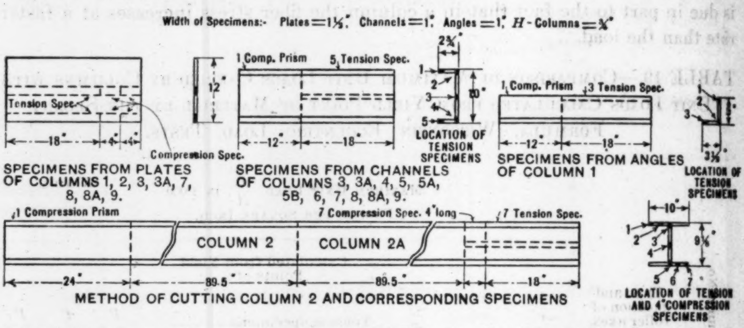
Kind of loading.	Column No.	Type and position of roller axes.	$\frac{l}{r}$.	$\frac{c}{r^2}$.	Maximum unit fiber stress in 1 000 lb. per sq. in., S.	YIELD POINT OF MATERIAL IN 1 000 LB. PER SQ. IN.			RATIOS.		
						Tension.		Compression.			
						Average for most strained fibers, Y_F	Average for entire column, Y_A.	Ultimate of whole prisms, Y_C.	$\frac{S}{Y_F}$	$\frac{S}{Y_A}$	$\frac{S}{Y_C}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Oblique	1		18	0.295	43.9	38.3	38.8	37.9	1.14	1.13	1.16
	2		21	0.271	42.9	36.1	36.1	36.5	1.19	1.19	1.18
	3		20	0.270	41.5	35.0	35.8	34.4	1.19	1.16	1.21
	3A		20	0.268	39.9	36.5	35.9	35.5	1.09	1.11	1.12
	4		22	0.206	36.8	32.0	32.6	32.2	1.15	1.18	1.14
	5		22	0.203	36.8	32.0	32.6	32.2	1.15	1.18	1.14
	5A		22	0.205	35.9	32.3	32.6	32.0	1.11	1.10	1.12
	6		22	0.206	38.7	32.0	32.6	32.2	1.21	1.19	1.20
Eccentric	5B		22	0.203	31.2	32.4	32.2	31.4	0.96	0.97	0.99
	8A		33	0.373	38.8	31.8	32.8	30.9	1.22	1.18	1.25
Eccentric	H 1		11	0.342	41.4	37.4	37.6	38.8	1.11	1.10	1.07
	H10B		12	0.340	43.1	37.4	38.0	37.6	1.15	1.13	1.15
	H 2		29	0.341	42.6	37.4	37.6	38.8	1.14	1.13	1.10
	H 3		50	0.342	42.5	37.4	37.6	38.8	1.14	1.13	1.09
	H 4		70	0.341	37.8	38.0	38.6	40.9	0.99	0.98	0.92
	H 5		90	0.339	36.2	36.4	36.7	35.3	0.99	0.99	1.02
Eccentric	H 1		20	1.06	53.6	39.0	37.6	38.8	1.37	1.42	1.38
	H 6		20	1.05	65.8	43.0	39.6	38.8	1.53	1.66	1.69
	H 7		51	1.04	69.0	48.0	39.6	38.8	1.60	1.74	1.78
	H 8		86	1.05	60.2	43.0	39.6	38.8	1.40	1.52	1.55
	H 9		121	1.02	51.6	45.3	41.8	39.4	1.14	1.23	1.31
	H10		158	1.06	44.0	41.8	38.0	37.6	1.05	1.16	1.17

In Table 19 are given values of observed and calculated ultimate strengths of columns tested eccentrically, the calculated values being based on three different values of yield point as given in Columns (6), (7), and (8) of the table. Columns (9), (10), and (11) give ratios of observed to calculated stresses. The results are relatively about the same as given in Table 18, except that the ratios of observed to calculated loads of Table 19 are somewhat less than the ratios of fiber stresses to yield points of Table 18; that is, the ultimate loads do not show the same relative excess as the ultimate fiber stresses. This is due in part to the fact that in a column the fiber stress increases at a faster rate than the load.

TABLE 19.—COMPARISON OF MAXIMUM UNIT LOADS CARRIED BY COLUMNS WITH UNIT LOADS CALCULATED FROM YIELD POINT OF MATERIAL BY THE SECANT FORMULA, WISCONSIN ECCENTRIC LOAD TESTS.

Column No.	Type and position of roller axes.	$\frac{l}{r}$.	$\frac{c}{r^2}$.	MAXIMUM UNIT LOAD, $\frac{P}{A}$, IN 1000 POUNDS PER SQUARE INCH.				RATIOS.		
				Observed, P.	Calculated from Yield Points of:					
					Tension specimens representing:		Whole compression prisms, P_C.			
					Average of most strained fibers, P_F.	Average for entire column, P_A.		$\frac{P}{P_F}$.	$\frac{P}{P_A}$.	$\frac{P}{P_C}$.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
H 1		11	0.342	20.7	18.5	18.6	19.2	1.12	1.11	1.08
H 10 *B*		12	0.340	21.0	18.5	18.8	18.6	1.14	1.12	1.13
H 2		29	0.341	20.0	18.1	18.2	18.7	1.10	1.10	1.07
H 3		50	0.342	18.0	17.0	17.1	17.6	1.06	1.05	1.02
H 4		70	0.341	15.1	15.6	15.8	16.6	0.97	0.96	0.91
H 5		90	0.339	12.6	13.4	13.5	13.1	0.94	0.93	0.96
H 1		20	1.06	26.9	18.1	18.2	18.8	1.48	1.48	1.43
H 6		20	1.05	28.9	20.9	19.3	18.9	1.38	1.50	1.53
H 7		51	1.04	21.7	18.9	17.6	17.3	1.16	1.24	1.27
H 8		86	1.05	16.2	15.2	14.4	14.2	1.07	1.12	1.14
H 9		121	1.02	10.6	11.8	11.4	11.0	0.90	0.93	0.96
H 10		158	1.06	7.8	8.2	7.9	7.8	0.95	0.99	1.00
5 *B*		22	0.203	19.0	20.0	19.8	19.4	0.95	0.96	0.98
8 *A*		33	0.373	18.0	15.3	15.6	14.9	1.18	1.15	1.21

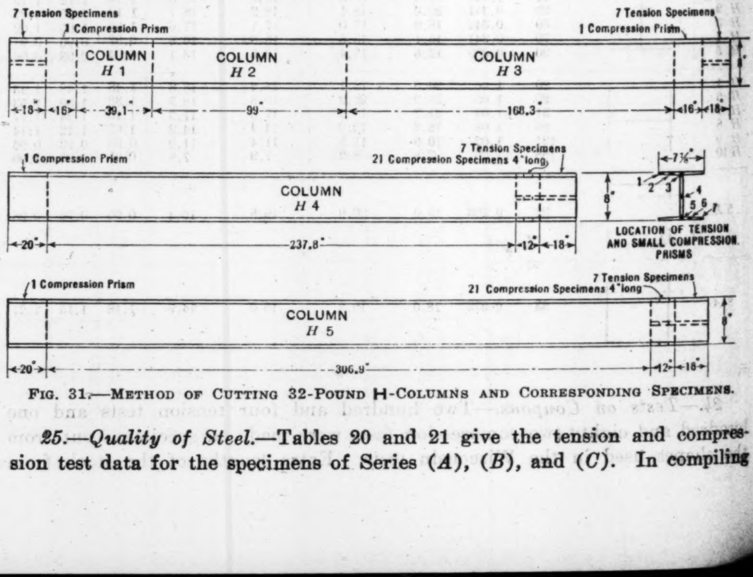
24.—Tests on Coupons.—Two hundred and four tension tests and one hundred and eighty-two compression tests were made on specimens cut from the shapes used in the Wisconsin tests. Extra lengths of the stock from which the columns were fabricated were furnished by the manufacturers with the columns. From these, test specimens were cut in accordance with the diagrams in Figs. 30 and 31. In these tests the proportional limit, yield point, ultimate strength, percentage elongation, percentage reduction in area, and modulus of elasticity were found. The speed of the movable head of the machine in the tension tests was 0.14 in. per min. as far as the yield point, and 0.023 to 0.068 per min. in the compression tests. From these tests on coupons only the strength data are presented herein.



Width of Specimens:- Plates = 1⅛". Channels = 1". Angles = 1". H - Columns = ¾"
Tension Spec.
Compression Spec.
18"
4"
4"
12"
SPECIMENS FROM PLATES OF COLUMNS 1, 2, 3, 3A, 7, 8, 8A, 9.
1 Comp. Prism
5 Tension Spec.
12"
18"
SPECIMENS FROM CHANNELS OF COLUMNS 3, 3A, 4, 5, 5A, 5B, 6, 7, 8, 8A, 9.
2⅝"
10"
1
2
3
4
5
LOCATION OF TENSION SPECIMENS
1 Comp. Prism
3 Tension Spec.
12"
18"
SPECIMENS FROM ANGLES OF COLUMN 1
1
2
3
3½"
LOCATION OF TENSION SPECIMENS
1 Compression Prism
7 Compression Spec. 4" long
7 Tension Spec.
COLUMN 2
COLUMN 2A
24"
89.5"
89.5"
18"
METHOD OF CUTTING COLUMN 2 AND CORRESPONDING SPECIMENS
10"
9⅛"
1
2
3
4
5
6
7
LOCATION OF TENSION AND 4" COMPRESSION SPECIMENS

FIG. 30.—LOCATION OF SPECIMENS IN COMPONENT PARTS OF COLUMNS OF SERIES (*A*) AND (*C*).

Columns Nos. *H*1 to *H*5 (Fig. 31) were cut at the University Shops from lengths of 8-in., 32-lb., H-sections, Heat No. 52*E*465. Columns Nos. *H*6 to *H*10 were cut at the mill, in similar manner, from Heat No. 39*F*273. No 4-in. compression prisms were made corresponding to Columns Nos. *H*9 and *H*10. All tension and short compression specimens were ¾ in. wide. The ends of all compression members were milled.



7 Tension Specimens
1 Compression Prism
COLUMN H 1
COLUMN H 2
COLUMN H 3
7 Tension Specimens
1 Compression Prism
8"
18"
16"
39.1"
99"
168.3"
16"
18"
1 Compression Prism
7 Tension Specimens
21 Compression Specimens 4" long
COLUMN H 4
8"
20"
237.8"
12"
18"
7⅛"
1
2
3
4
5
6
LOCATION OF TENSION AND SMALL COMPRESSION PRISMS
1 Compression Prism
7 Tension Specimens
21 Compression Specimens 4" long
COLUMN H 5
8"
20"
306.9"
12"
18"

FIG. 31.—METHOD OF CUTTING 32-POUND H-COLUMNS AND CORRESPONDING SPECIMENS.

25.—Quality of Steel.—Tables 20 and 21 give the tension and compression test data for the specimens of Series (*A*), (*B*), and (*C*). In compiling

TABLE 20.—AVERAGE STRENGTH PROPERTIES OF MATERIAL IN COLUMNS OF SERIES (*A*) AND (*C*).

Column No.	Shape.	Location of specimen.	Number of Specimens Tested at University of Wisconsin.		Tension, in 1 000 Pounds per Square Inch.					Compression, in 1 000 Pounds per Square Inch.	
			Tension.	Compression.	Proportional limit.	Yield Point.		Ultimate Strength.		Proportional limit.	Ultimate strength or yield point.
						University of Wisconsin.	Mill.*	University of Wisconsin.	Mill.*		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	5 by 3½-in. angle	Toe	4	......	33.9	38.5	......	60.0	......	......	......
	5 by 3½-in. angle	Average	6	2	33.5	38.3	37.1	60.1	60.5	34.0	37.9†
	10 by 5/16-in. plate	Edge	2	5	37.8	41.6	......	63.4	......	39.0	41.8
	10 by 5/16-in. plate	Average	4	10	37.9	40.8	36.1	63.4	60.1	37.6	39.8
2	10-in. 50-lb. H-column	Toes	2	......	36.0	37.0	......	58.8	......	32.6	36.6
		Flanges	6	......	33.7	36.1	......	60.1	......	32.2	35.7
		Average	7	1	33.7	36.1	39.9	60.0	60.0	25.0	36.5†
3	10-in. 15-lb. channel	Flanges	8	......	26.6	34.4	......	62.0	......	......	......
	10-in. 15-lb. channel	Average	10	2	29.4	36.4	36.2	62.4	61.5	22.5	34.8†
	12 by ¼-in. plate	Average	4	8	33.2	35.0	34.0	55.2	56.2	29.6	34.4
3*A*	10-in. 15-lb. channel	Flanges	8	......	31.2	35.5	......	57.4	......	......	......
	10-in. 15-lb. channel	Average	10	2	30.5	35.5	36.2	57.3	61.5	25.0	33.4†
	12 by ¼-in. plate	Average	4	8	33.4	36.5	34.7	58.2	57.4	31.0	35.5
4, 5, 6,	10-in. 20-lb. channel	Web and root	18	......	26.4	32.0	......	54.0	......	......	......
	10-in. 20-lb. channel	Average	30	5	27.1	32.6	34.6	54.2	56.8	23.0	32.2†
4	2¼ by 5/16-in. lacing-bar	......	......	......	......	......	37.4	......	64.1	......	......
	¼-in. stay-plate	......	......	......	......	......	34.0	......	56.2	......	......
5	2½ by ¼-in. lacing-bar	......	......	......	......	......	36.8	......	61.3	......	......
	¼-in. stay-plate	......	......	......	......	......	34.0	......	56.2	......	......
6	2½ by ⅜-in. lacing-bar	......	......	......	......	......	35.6	......	62.4	......	......
	⅜-in. stay-plate	......	......	......	......	......	36.3	......	60.4	......	......
5*A*	10-in. 20-lb. channel	Web and root	6	......	30.0	32.3	......	52.0	......	......	......
	10-in. 20-lb. channel	Average	10	2	30.0	32.6	34.6	52.4	56.8	26.0	32.0†
	2¼ by ¼-in. lacing-bar	......	......	......	......	......	36.8	......	61.3	......	......
	¼-in. stay-plate	......	......	......	......	......	34.7	......	57.4	......	......
5*B*	10-in. 20-lb. channel	Web and root	6	......	25.3	32.4	......	53.4	......	......	......
	10-in. 20-lb. channel	Average	10	......	25.1	32.2	34.6	53.6	56.8	24.5	31.4†
	2¼ by ¼-in. lacing-bar	......	......	......	......	......	36.8	......	61.3	......	......
	¼-in. stay-plate	......	......	......	......	......	34.7	......	57.4	......	......
7	10-in. 20-lb. channel	Web and root	6	......	27.4	32.7	......	52.6	......	......	......
	10-in. 20-lb. channel	Average	10	2	28.1	32.9	34.6	52.8	56.8	26.2	33.6†
	12 by ¼-in. plate	Average	4	8	35.6	37.2	34.7	58.8	57.4	31.2	35.1
8	10-in. 20-lb. channel	Web and root	6	......	28.4	32.2	......	52.4	......	......	......
	10-in. 20-lb. channel	Average	9	2	27.9	32.4	34.6	52.7	56.8	25.0	33.6†
	12 by ¼-in. plate	Average	4	8	31.8	37.2	34.7	58.3	57.4	31.3	36.0
8*A*	10-in. 20-lb. channel	Root and toe	8	......	29.1	31.8	......	52.4	......	......	......
	10-in. 20-lb. channel	Average	10	......	28.6	32.4	34.6	53.2	56.8	24.8	30.9†
	12 by ¼-in. plate	Average	2	4	31.8	38.7	34.7	62.0	57.4	......	38.2
9	10-in. 20-lb. channel	Web and root	6	......	27.8	31.7	......	51.8	......	......	......
	10-in. 20-lb. channel	Average	10	2	27.9	32.2	34.6	52.2	56.8	25.0	31.4†
	12 by ¼-in. plate	Average	4	8	33.1	36.0	34.7	58.7	57.4	31.6	35.8

* Heat test data.

† Ultimate strength of a full-sized prism.

the averages for the different shapes, the results were weighted in proportion to the volume of the shape tributary to the specimen.

Study of the tabulated data obtained at Wisconsin shows that all shapes passed the specification requirements for yield point, but the average tensile strength for all the 10-in., 20-lb. channels was below requirements. The variations in the different properties are given in Table 22. Attention is called to the similarity in range for tensile and compressive yield point, also to the wide range of proportional limit values. It will also be noted that the thinner sections of a given shape exhibited higher yield points than the thicker sections.

TABLE 21.—AVERAGE STRENGTH PROPERTIES OF STEEL IN 8-INCH H-COLUMNS OF SERIES (*B*).

Column No.	Location of specimens.	NUMBER OF SPECIMENS TESTED AT UNIVERSITY OF WISCONSIN.		TENSION, IN 1 000 POUNDS PER SQUARE INCH.					COMPRESSION, IN 1 000 POUNDS PER SQUARE INCH.	
					Yield Point.		Ultimate Strength.			
		Tension.	Compression.	Proportional limit.	University of Wisconsin.	Mill.*	University of Wisconsin.	Mill.*	Proportional limit.	Ultimate strength of full-sized prisms.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
H 1, *H* 2 and *H* 3	Toes	4	..	36.0	39.0	....	61.1	....	....	....
	Flanges	12	..	33.6	37.4	....	60.8	....	....	....
	Ave.	14	2	34.2	37.6	43.7	61.0	64.3	27.0	38.8
H 4	Toes	2	..	34.8	39.6	....	60.4	....	....	....
	Flanges	6	..	33.6	38.0	....	61.2	....	....	....
	Ave.	7	1	35.2	38.6	43.7	61.7	64.3	26.0	40.9
H 5	Toes	2	..	34.0	37.6	....	58.2	....	....	....
	Flanges	6	..	33.1	36.4	....	58.3	....	....	....
	Ave.	7	1	33.9	36.7	43.7	58.4	64.3	28.5	35.3
H 6, *H* 7 and *H* 8	Toes	4	..	39.7	43.0	....	62.9	....	....	....
	Flanges	12	..	34.6	39.4	....	60.5	....	....	....
	Ave.	14	2	34.4	39.6	42.2	60.6	60.9	26.3	38.8
H 9	Toes	2	..	42.6	45.3	....	65.8	....	....	....
	Flanges	6	..	39.0	41.3	....	64.7	....	....	....
	Ave.	7	2	39.7	41.8	42.2	64.2	60.9	25.1	39.4
H 10 and *H* 10 *B*	Toes	2	..	34.8	41.8	....	61.1	....	....	....
	Flanges	6	..	30.9	37.4	...	59.0	....	....	....
	Ave.	7	2	32.4	38.0	42.2	58.9	60.9	25.6	37.6

* Heat test data.

Mill *vs.* Laboratory Tests.—Averaging the laboratory tests for a given shape and heat and comparing with mill tests, the maximum, average, and minimum ratios of

$$\frac{\text{Mill yield point}}{\text{Laboratory yield point}}$$

for the nine heats are found to be 1.16, 1.02, and 0.88, respectively. Proceeding in a similar manner, the corresponding ratios of

$$\frac{\text{Mill ultimate strength}}{\text{Laboratory ultimate strength}}$$

are, respectively, 1.07, 1.00, and 0.95.

Comparison of Yield Points in Tension and Compression.—Although the results in Table 22 indicate a considerable range in yield points of the material in the different shapes, the data of Table 23 shows that the ratio,

$$\frac{\text{Average yield point in tension}}{\text{Average yield point in compression}}$$

was nearly constant. For the nine heats represented, this ratio varied from 0.99 to 1.05, and averaged 1.01. These data supplement and confirm the statement previously advanced,[9] that, in general, the tensile yield point is a close indication of the compressive yield point, and, for the purpose in question, probably as satisfactory an indication of the column strength.

TABLE 22.—VARIATIONS IN STRENGTH PROPERTIES OF SPECIMENS IN WISCONSIN TESTS.

Property tested. (1)	Kind of stress. (2)	Highest average stress, in 1 000 lb. per sq. in. (3)	Shape having highest stress. (4)	Lowest average stress, in 1 000 lb. per sq. in. (5)	Shape having lowest stress. (6)	Ratio, Highest-lowest / Specified minimum in percentage. (7)
Proportional limit..	Tension	39.7	8-in. 32-lb. H-column	25.1	10-in. 20 lb. channel	49*
Yield point.........	Tension	41.8	8 in 32-lb. H-column	31.7	10-in. 20-lb. channel	33
Ultimate strength..	Tension	64.2	8 in. 32-lb. H-column	52.2	10-in. 20-lb. channel	22
Proportional limit..	Compression	37.6	10 by $\frac{5}{16}$-in. plate	22.5	10-in. 15-lb. channel	50*
Yield point.........	Compression	40.9	8-in. 32-lb. H-column	30.9	10-in. 20 lb. channel	33*

* No minimum specified—based on 30 000 lb. per sq. in.

26.—Effect of Location on Strength of Specimens.—Table 24 furnishes information regarding the effect of the position of the test piece in the shape on the yield point and ultimate strength. For the H-columns the specimens cut from the intermediate portions of the flanges[10] appear to represent best the entire shape. They also were the least variable. For the channels the web specimens were most representative. For the plates there was not much difference (2%) between specimens cut from the edge and from the center, although the former were consistently the stronger. The number of specimens from angles is too small to warrant definite conclusions. The root specimens were the best representatives for the angles tested. In most cases the specimens cut from the toes or edges of shapes exhibited higher yield points than other specimens cut from the same shapes.

27.—The Uniformity of the Modulus of Elasticity of the Wisconsin Test Columns.—By plotting the average strains for gauge lines which were symmetrically disposed about the axis of a given column, the modulus of elasticity was computed for the mid-segments of all Wisconsin test columns and in most cases for the segments at or near the quarter-points. Owing to the effect of

---

[9] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1524.

[10] See similar results in *Technologic Paper No. 328*, U. S. Bureau of Standards.

irregularities in milling of the ends of columns and in the bearing blocks, the average moduli for the gauge lines nearest the column ends are omitted from this discussion. The results of these calculations appear in Tables 25 and 26. In calculating E for the segments of the built-up columns, the strains for the four corner gauge lines were averaged; but, in similar calculations for the H-columns, strains along the eight gauge lines shown in Fig. 8 were usually averaged. Values of mean E for the column ("column-E") were obtained by averaging the values of E for the segments.

TABLE 23.—RELATIONSHIP BETWEEN YIELD POINTS IN TENSION AND COMPRESSION FOR SPECIMENS CUT FROM COLUMNS OF SERIES (*A*), (*B*), AND (*C*).

Shape.	Number of				Average Yield Point in 1 000 Pounds per Square Inch.		Ratio, $\frac{T}{C}$
	Heat.	Pieces sampled.	Specimens Tested in.				
			Tension.	Compression.	Tension, *T*.	Compression, *C*.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
8-in. 32-lb. H-column .	52*E*465 and 39*F*273	6	56	10*	38.8	38.5	1.01
8-in. 32-lb. H-column ...	52*E*465	2	14	42	37.6	38.0	0.99
10-in. 50-lb. H-column	33*E*287	1	7	7	36.1	36.0	1.00
	33*E*287	..	..	1*	....	36.5	0.99
10-in. 15-lb. channel...	13291	2	..	10	35.5†	33.8	1.05
	13291	4	20	4*	36.0	34.1	1.05
10-in. 20-lb. channel,..	21244	8	..	39	32.5†	31.6	1.03
	21244	18	89	17*	32.5	32.2	1.01
12 by ¼-in. plate......	11010 and 904905	11	22	44	36.6	35.8	1.02
10 by 5/16-in. plate.......	925696	2	4	10	40.8	39.8	1.03
5 by 3½ by 5/16-in. angle.	48182	2	6	2*	38.3	37.9	1.01

* Full-sized prisms.

† Averages corresponding to those in Column (7).

It should be observed that practically all the columns were subjected to several trial loadings prior to the determination of the strains used in these calculations. This condition contributed to the obliteration of initial sets and to somewhat higher moduli of elasticity than would have been obtained had it been possible to have made all measurements during an initial loading.

The data in Tables 25 and 26 show that the average value of "column-E" for the 8-in. gauge lines of both the built-up and H-columns averaged 29 800 000 lb. per sq. in.; also, that "column-E" for the 20-in. gauge lines averaged 29 400 000 and 29 800 000 lb. per sq. in. for the respective types of columns. Considering 8-in. gauge lines, Column No. 1 had the lowest "column-E", 27 800 000 lb. per sq. in., and Columns Nos. 7 and *H*5 the highest, 30 900 000 lb. per sq. in. Hence, the range in "column-E" for the built-up columns is 10%; whereas, for the H-columns, it is 7 per cent.

Since marked variation in values of E within a column materially affects its strength, the percentage range in E for the three segments of each column were inserted in Columns (8) of Tables 25 and 26 to secure some data covering this point. These values were calculated from,

$$\frac{\text{maximum } E\text{—minimum } E}{\text{“column-}E\text{”}}$$

The average range of E for the built-up columns was 5.8% and for the H-columns the range was 3.9 per cent. The greatest range, 9.8%, occurred in batten-plate column (No. 9).

TABLE 24.—RELATIONSHIP BETWEEN TENSILE STRENGTH PROPERTIES OF SPECIMENS CUT FROM DIFFERENT PARTS OF STRUCTURAL SHAPES, SERIES (*A*) TO (*C*).

Shape.	Location of specimen.	Number of specimens tested.	YIELD POINT.				ULTIMATE STRENGTH.		
			Mean in 1 000 lb. per sq. in.	Ratio to whole.	Average Percentage Variation from		Mean in 1 000 lb. per sq. in.	Average Percentage Variation from	
					Mean. ‡	Grand mean. †		Mean. ‡	Grand mean. †
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
8-in., 32-lb. H-column...	Toe	16	41.0	1.06	5.7	6.9	61.7	3.5	3.1
	Flange	16	38.2	0.99	5.0	5.0	60.5	2.9	3.0
	Root	16	36.2	0.94	4.7	7.0	60.0	4.2	4.0
	Web	8	40.5	1.04	3.9	5.1	61.6	3.9	4.0
	Whole	8	38.8*	1.00	....	6.2	60.8*	....	3.5
10-in., 15-lb. channel....	Toe	8	34.7	0.98	4.0	3.9	59.5	5.1	3.1
	Root	8	35.1	0.99	4.1	3.8	59.8	3.6	2.1
	Web	4	37.4	1.06	5.6	5.9	60.2	5.0	2.5
	Whole	4	36.0*	1.00	....	4.3	59.9*	....	2.4
10-in., 20-lb. channel....	Toe	35	33.9	1.04	4.4	5.7	54.5	1.7	3.1
	Root	36	31.4	0.97	3.6	4.8	52.5	0.9	2.1
	Web	18	32.8	1.01	1.9	1.9	53.4	2.5	2.5
	Whole	18	32.5*	1.00	....	4.6	53.3*	....	2.4
12 by ¼-in. plate........	Edge	11	36.9	1.01	3.0	3.1	59.1	2.5	2.5
	Center	11	36.3	0.99	2.4	2.4	59.0	2.2	2.2
	Whole	11	36.6*	1.00	....	2.7	59.0*	....	2.3
5 by 3 by 5/16-in. angle..	5-in. toe	2	36.4	0.95	0.3	5.5	59.7	0.5	0.7
	3-in. toe	2	41.2	1.08	2 1	6.8	60.4	0.5	0.5
	Root	2	38.0	0.99	0.7	1.4	60.3	0.5	0.5
	Whole	2	38.3*	1.00	....	4.6	60.2*	....	0.6

*Calculated by weighting specimens in proportion to tributary area.

† Mean of all specimens from the type of shape indicated in Column (1).

‡ Mean of specimens from the location indicated in Column (2).

If one assumes E equal to 30 000 000 lb. per sq. in. for calculations in connection with these columns, the results entered in Columns (9) of Tables 25 and 26 show the average errors in percentage for the different segments and gauges of these columns. For the built-up columns the average percentage variation of E from 30 000 000 lb. per sq. in. is only 3.2; for the H-columns, it is only 2.1. The maximum variation, 8.7%, is for Column No. 3*A*.

When one takes into account the nature of strain-gauge measurements and the variability of the cross-sectional area of such members, these data

TABLE 25.—VALUES OF MODULUS OF ELASTICITY FOR DIFFERENT PORTIONS OF BUILT-UP COLUMNS AS FOUND BY STRAIN-GAUGE MEASUREMENTS IN WISCONSIN CENTER LOAD TESTS.

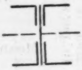
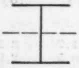
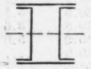
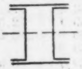
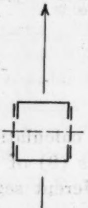
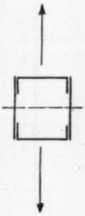
Column No.	Type and position of roller axes.	Position of gauged portion.	Number of gauge lines averaged.	Length of gauge, in inches.	AVERAGE MODULUS OF ELASTICITY, *E*, IN 1 000 000 POUNDS PER SQUARE INCH, FOR		Range in *E*, in percentage (8-in. gauges).	Percentage variation from 30 000 000 lb. per sq. in.
					Gauged portion.	Column (8-in. gauges).		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1		Middle	4	8	27.8	27.8	1.5	7.7
		Middle	4	20	28.2	....	....	6.0
2		Top quarter	4	8	30.0	29.7	2.7	0.0
		Middle	4	8	29.2	....	....	2.7
		Bottom quarter	4	8	29.9	....	....	0.3
		Middle	4	20	31.5	....	....	5.0
3		Middle	4	8	30.4	30.4	....	1.3
		Middle	4	20	28.6	....	....	4.7
3*A*		Middle	4	8	27.9	27.9	....	7.0
		Middle	4	20	27.4	....	....	8.7
4		Top quarter	4	8	30.2	30.0	7.0	0.7
		Middle	4	8	28.9	....	....	3.7
		Bottom quarter	4	8	31.0	....	....	3.3
		Middle	4	20	29.9	....	....	0.3
5		Top quarter	4	8	31.6	30.6	6.2	5.3
		Middle	4	8	30.6	....	....	2.0
		Bottom quarter	4	8	29.7	....	....	1.0
5*A*		Top quarter	4	8	31.8	30.0	9.7	6.0
		Middle	4	8	28.9	....	....	3.7
		Bottom quarter	4	8	29.4	....	....	2.0
		Middle	4	20	28.9	....	....	3.7
5*B*		Top quarter	4	8	31.2	30.5	4.9	4.0
		Middle	4	8	29.7	....	....	1.0
		Bottom quarter	4	8	30.6	....	....	2.0
6		Top quarter	4	8	30.1	30.6	2.6	0.3
		Middle	4	8	30.9	....	....	3.0
		Bottom quarter	4	8	30.7	....	....	2.3
		Middle	4	20	30.1	....	....	0.3

TABLE 25.—(*Continued.*)

Column No.	Type and position of roller axes.	Position of gauged portion.	Number of gauge lines averaged.	Length of gauge, in inches.	AVERAGE MODULUS OF ELASTICITY, *E*, IN 1 000 000 POUNDS PER SQUARE INCH, FOR		Range in *E*, in percentage (8-in. gauges).	Percentage variation from 30 000 000 lb. per sq. in.
					Gauged portion.	Column (8-in. gauges).		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
7		Top quarter	4	8	29.6	30.9	7.4	1.3
		Middle	4	8	31.9	....	....	6.3
		Bottom quarter	4	8	31.2	....	....	4.0
		Middle	4	20	29.4	....	....	2.0
8		Top quarter	4	8	28.7	28.6	0.7	4.3
		Middle	4	8	28.5	....	....	5.0
		Bottom quarter	4	8	28.7	....	....	4.3
		Middle	4	20	29.7	....	....	1.0
8A		Top quarter	4	8	30.2	29.9	6.7	0.7
		Middle	4	8	28.8	....	....	4.0
		Bottom quarter	4	8	30.8	....	....	2.7
9		Top quarter	4	8	30.8	30.7	9.8	2.7
		Middle	4	8	32.2	....	....	7.3
		Bottom quarter	4	8	29.2	....	....	2.7
		Middle	4	20	30.2	....	....	0.7
Average for 8-in. gauge lines						29.8	5.8	3.2*
Average for 20-in. gauge lines						29.4	....	3.2*
Average for all gauge lines						29.6	....	3.2*

* Calculated by averaging the appropriate results in Column (9).

furnish convincing proof of the uniformity of the modulus of elasticity of steel within a given shape and as between shapes.

Values of *E* from tests on the coupons cut out of these different shapes exhibit similar uniformity, but the average values of the tension and compression moduli from the **H**-column coupon tests, approximately 29 200 000 and 29 000 000 lb. per sq. in., respectively, are somewhat less than the values found in the column tests.

(D) COLUMN THEORIES: THE COLUMN IN A TRUSS

It is not intended to give herein any considerable discussion of the various column formulas, but only a brief review of the various types and some points relative to their derivation and applicability.

28.—The Euler Formula.—This well-known formula for various end conditions has been shown by many tests conducted with carefully designed end conditions to be correct and reliable under the conditions assumed. It is based, as is well known, on the elastic modulus of the material and not on its strength. It is, therefore, applicable, if at all, only to lengths in which buckling can occur within the elastic limit of the material. It has proven very serviceable for such slender columns as airplane struts, but in the general field of structural engineering, it serves mainly as a guide for the limiting value, for the longer lengths, in the study of tests of columns and of column formulas, where conditions may approach those assumed in the Euler formula.

TABLE 26.—VALUES OF MODULUS OF ELASTICITY FOR DIFFERENT PORTIONS OF H-COLUMNS AS FOUND BY STRAIN-GAUGE MEASUREMENTS IN WISCONSIN CENTER LOAD TESTS.

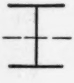
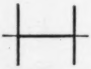
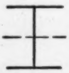
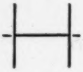
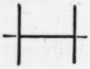
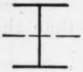
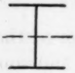
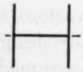
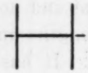
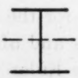
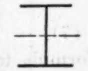
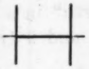
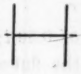
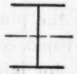
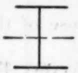
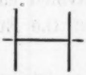
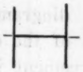
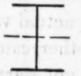
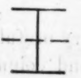
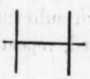
Column No. (1)	Position of roller axes. (2)	Position of gauged portion. (3)	Number of gauge lines, average. (4)	Length of gauge, in inches. (5)	AVERAGE MODULUS OF ELASTICITY, E, IN 1 000 000 POUNDS PER SQUARE INCH FOR: Gauged portion. (6)	AVERAGE MODULUS OF ELASTICITY, E, IN 1 000 000 POUNDS PER SQUARE INCH FOR: Column (8-in. gauges). (7)	Range in E, in percentage (8-in. gauges). (8)	Percentage variation from 30 000 000 lb. per sq. in. (9)
H 1		Middle	8	8	30.4	30.4	....	1.3
		Middle	8	8	29.7	29.7	....	1.0
H 6		Middle	4	8	29.8	29.8	....	0.7
		Middle	4	8	29.2	29.2	....	2.7
H 2		Top quarter	8	8	29.8	29.6	5.1	0.7
		Middle	6	8	30.2			0.7
		Bottom quarter	8	8	28.7			4.3
		Top quarter	8	8	29.3	29.1	1.7	2.3
		Middle	6	8	29.2			2.7
		Bottom quarter	8	8	28.8			4.0
H 7		Top quarter	8	8	29.4	29.6	1.7	2.0
		Middle	6	8	29.9			0.3
		Bottom quarter	8	8	29.4			2.0
		Top quarter	8	8	28.9	28.8	1.0	3.7
		Middle	6	8	28.9			3.7
		Bottom quarter	8	8	28.6			4.8
H 8		Top quarter	..	..	....	30.4	6.6	....
		Middle	6	8	29.4			2.0
		Bottom quarter	8	8	31.4			4.7
		Middle	4	20	29.9			0.3
		Top quarter	7	8	....	29.6	4.1	....
		Middle	6	8	29.0			3.3
		Bottom quarter	7	8	30.2			0.7
		Middle	4	20	30.0			0.0

TABLE 26.—(*Continued.*)

Column No.	Position of roller axes.	Position of gauged portion.	Number of gauge lines, average.	Length of gauge, in inches.	AVERAGE MODULUS OF ELASTICITY, *E*, IN 1 000 000 POUNDS PER SQUARE INCH FOR: Gauged portion.	Column (8-in. gauges).	Range in *E*, in percentage (8-in. gauges).	Percentage variation from 30 000 000 lb. per sq. in.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
H 8		Top quarter	8	8	29.1			3.0
		Middle	6	8	29.2			2.7
		Bottom quarter	8	8	30.0	29.4	3.1	0.0
		Middle	4	20	29.8			0.7
		Top quarter	8	8	28.7			4.3
		Middle	6	8	29.7			1.0
		Bottom quarter	8	8	30.7	29.7	6.7	2.3
		Middle	4	20	30.2			0.7
H 4		Top quarter	8	8	31.4			4.7
		Middle	6	8	31.4			4.7
		Bottom quarter	8	8	29.7	30.8	5.5	1.0
		Middle	4	20	29.8			0.7
		Top quarter	8	8	29.0			3.3
		Middle	6	8	29.2			2.7
		Bottom quarter	8	8	29.5	29.2	1.7	1.7
		Middle	4	20	30.0			0.0
H 9		Top quarter	8	8	30.4			1.3
		Middle	6	8	29.7			1.0
		Bottom quarter	8	8	29.7	29.9	2.3	1.0
		Middle	4	20	29.5			1.7
		Top quarter	8	8	30.4			1.3
		Middle	6	8	29.2			2.7
		Bottom quarter	8	8	29.2	29.6	4.1	2.7
		Middle	4	20	29.0			3.3
H 5		Top quarter	..	..	....			....
		Middle	6	8	29.5			1.7
		Bottom quarter	8	8	32.3	30.9	9.1	7.7
		Middle	4	20	30.1			0.3
		Top quarter	8	8	30.5			1.7
		Middle	6	8	30.0			0.0
		Bottom quarter	8	8	30.9	30.5	3.0	3.0
		Middle	4	20	30.2			0.7
H 10		Top quarter	8	8	30.4			1.3
		Middle	6	8	30.6			2.0
		Bottom quarter	8	8	29.8	30.3	2.6	0.7
		Middle	4	20	29.3			2.3
		Top quarter	8	8	28.3			5.7
		Middle	6	8	29.4			2.0
		Bottom quarter	8	8	29.3	29.0	3.8	2.3
		Middle	4	20	30.3			1.0
Average for 8-in. gauge lines						29.8		2.4*
Average for 20-in. gauge lines						29.8	3.9	1.0*
Average for all gauge lines						29.8		2.1*

* Calculated by averaging the appropriate results in Column (9).

For pivoted ends, the formula is,

$$P/A = \frac{\pi^2 E}{\left(\frac{l}{r}\right)^2} \quad \text{..............................(7)}$$

For rigidly fixed ends, the formula becomes,

$$P/A = \frac{4\,\pi^2 E}{\left(\frac{l}{r}\right)^2} \quad \text{..............................(8)}$$

or the same result as by using the half length in Equation (7).

In some respects, it is helpful, in applying a column formula to a particular case, to estimate, from a knowledge of end conditions, the probable points of inflection and then to use the resulting "free length" in a formula for pivoted ends. Thus, in the tests plotted in Figs. 7, 8, and 9 (report of 1926[11]), the flat-end tests appear to agree, for high values of $\frac{l}{r}$, with the Euler curve for fixed ends for a value of 1.12 l, or for a free length of 0.56 l; and the pin-end tests, for a value of 0.78 l; that is, in the case of the flat-end tests, the points of inflection were about 0.56 l apart, and for the pin-end tests about 0.78 l apart; the flat-end columns were not quite fixed; and the pin-end columns were about midway between theoretically fixed and pivoted conditions.

Whatever the column formula adopted, it is obvious that, for large values of $\frac{l}{r}$, the formula should give results somewhere between those of the Euler formulas for pivoted and for fixed ends; for columns with very little end rigidity, values nearly as low as the Euler formula for pivoted ends; and for columns well fixed, values somewhat less than those of the Euler formula for fixed ends.

29.—Karman's and Engesser's Variations.—These formulas attempt to use the Euler method and apply it to the actual stress-strain diagram of the material beyond its elastic limit. They also take account of the effect of form of section. This is a rational method, and fair agreement is found between theory and test results; but to use it in practice requires the assumption of a standard stress-strain diagram of the material. The actual variations from the average in this respect and the variations due to other causes leads to the conclusion that the application of these refinements is not warranted in practice.

30.—The Theoretical Formula for Eccentric Loads.—A very useful formula for many purposes is the theoretical formula for pivoted-end columns with loads applied with a known eccentricity, the same at both ends. Inasmuch as considerable use will be made of formulas of this type in subsequent discussions, it will be convenient to list herein the notation used, repeating part of that already used:

p = P/A = average compressive stress per unit area of column;
e = eccentricity;

---

[11] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), pp. 1527–1529.

c = distance from gravity axis to extreme fiber;

$\frac{e\,c}{r^2}$ = eccentric ratio;

$p_e = \frac{\pi^2 E}{\left(\frac{l}{r}\right)^2}$ = Euler load per square inch for centrally loaded pivoted-end columns;

$\phi = \frac{\pi}{2}\sqrt{\frac{p}{p_e}}$, a function of frequent occurrence;

$q = \sqrt{\frac{P}{E\,I}}$;

f_b = intensity of bending stress on extreme fiber at center;

f = total unit stress on extreme fiber at center on concave side.

Referring to Fig. 32, the equation of the elastic line is,

$$y = e\,(\tan\phi \sin q\,x + \cos q\,x - 1)$$

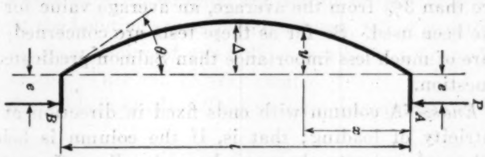


FIG. 32.

The inclination at the end is,

$$\frac{d\,y}{d\,x}(x = 0) = \theta = e\,q \tan\phi \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

$$f_b = p\,\frac{e\,c}{r^2}\sec\phi$$

The total stress is,

$$f = p + f_b = p\left(1 + \frac{e\,c}{r^2}\sec\phi\right)$$

and,

$$p = \frac{f}{1 + \frac{e\,c}{r^2}\sec\phi} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Placing f = yield-point strength of the material, the value of p will approximate the ultimate strength of the column.

If the loads are applied centrally, but the column is initially bent to an amount equal to e at the center, the resulting stresses are somewhat less than for an eccentricity of loading of the same amount, but the difference is not great enough to warrant special treatment of this case. Where there is both eccentricity of load and crookedness of column, both eccentricities may be combined with little error.[12]

---

[12] For full discussion, see work on "Columns," by E. H. Salmon, Oxford Technical Publications, pp. 29, 42.

The foregoing formula, frequently known as the "secant" formula, is satisfactory from a theoretical standpoint, and for analytical purposes has many advantages over others; but for practical use, the eccentricity, e, must be assumed or determined by tests or observations, in much the same manner as the constants in other formulas, and when this is done, the formula is not readily solved for particular values of $\frac{l}{r}$, as the value of p is included also in the quantity, ϕ. This objection can be removed by the use of tables, but there is something to be said in favor of a formula that can be readily applied to any particular case. Various approximations to the secant formula have been proposed, the most elaborate study known to the Committee being that by Mr. R. W. Hawken,[13] in which he analyzes about seven different approximations and proposes one of his own.[14]

The accuracy of the secant formula when the eccentricity is known is well indicated by the results of the eccentric load tests of Series (C). The value of E used in each case was that for the column in question, but as these values varied not more than 3% from the average, an average value for this quantity might well have been used. So far as these tests are concerned, therefore, the irregularities are of much less importance than Salmon predicated in his treatment of this question.

31.—Fixed Ends.—A column with ends fixed in direction at all loads can have no eccentricity of loading; that is, if the column is held securely in alignment by the end supports, there can be no bending effect at the ends due to eccentricity of loading. As a matter of fact, most columns rigidly connected in a framework are not really fixed-end columns, but are subjected to a certain amount of end moment and rotation of axis at the end, due to the action of the framework as a whole. This action will be discussed subsequently; but such a column is not a fixed-end column. The only kind of eccentricity which can exist in a fixed-end column is that (1) due to crookedness or curvature of the

---

[13] "Column Analysis and Design," Univ. of Queensland, Sydney, 1918; see, also, "A Simplified Column Formula of the Secant Type," by J. B. Hunley, *Bulletin No. 300,* Am. Ry. Eng. Assoc., 1927.

[14] The approximate formula proposed by Hawken is,

$$p = \frac{f}{1 + \frac{e\,c}{r^2} \cdot \frac{1 + 0.25\,\frac{p}{p_e}}{1 - \frac{p}{p_e}}}$$

in which, $\sec \frac{\pi}{2} \sqrt{\frac{p}{p_e}}$ of Equation (10) is replaced by the quantity, $\frac{1 + 0.25\,\frac{p}{p_e}}{1 - \frac{p}{p_e}}$. This is remarkably close to the true value, giving results for p in error by less than 1% for values of $\frac{p}{p_e}$ as great as 0.9. This equation is readily solvable for p_e, giving,

$$p_e = p + \frac{\frac{f}{p} - \left(1 - 0.25\,\frac{e\,c}{r^2}\right)}{\frac{f}{p} - \left(1 + \frac{e\,c}{r^2}\right)}$$

and from the relation, $\left(\frac{l}{r}\right)^2 = \frac{\pi^2\,E}{p_e}$, various values of $\frac{l}{r}$ can be directly calculated for various values of p, and curves or tables prepared for various values of $\frac{e\,c}{r^2}$.

axis of resistance; (2) due to actual crookedness of the section, or variation in the section; or (3) due to variation in the modulus of elasticity. The effect of a given amount of initial curvature can be determined by the same general process as applied to the eccentrically loaded, pivoted-end column. The mathematical process is somewhat complicated, and it will be sufficient to give here the general results as contained in Salmon's analysis. He shows that the bending stresses in such a case, assuming parabolic initial curvature, are, for small values of $\frac{p}{p_e}$ $\left(\text{low loads, or low values of } \frac{l}{r}\right)$, about twice as great at the end as at the center, like the bending stresses in an ordinary beam fixed at the ends. The sum of these end and center stresses is somewhat less than the center fiber stress in a pivoted end column of half the length, with the same initial eccentricity. For higher loads, the center and end stresses approach equality, and the sum of the two differs by an increasing amount from the fiber stress in a pivoted-end column of half length. Considering all the conditions, it may be concluded that the effect of eccentricity due to initial bends in a truly fixed-end column, on the bending stress in the column, is not greatly different from the effect of an eccentricity of one-half as much on a pivoted-end column of one-half the length. This will be very small.

32.—The Rankine Formula.—This well-known equation is of the form,

$$p = \frac{f}{1 + K\left(\frac{l}{r}\right)^2} \quad \text{.......................(11)}$$

in which, K is a constant depending on the end conditions and rests on experimental determination. For the ultimate strength, f is approximately the yield point of the material.

A rather serious objection to this formula is that, in the nature of the case, K can hardly be a constant, as may readily be shown. The formula can be written, $f = p + p\,K\left(\frac{l}{r}\right)^2$, in which the term, $p\,K\left(\frac{l}{r}\right)^2$, represents the bending fiber stress, f_b, at the center. Also, necessarily, $f_b = p\,\frac{\Delta\,c}{r^2}$, in which, Δ equals the total eccentricity at the center of the column. If this eccentricity is all due to bending, for any beam, $\Delta = \frac{K'\,f_b\,l^2}{c}$, in which, K' is a constant depending on the manner of loading. $\left(\text{For a uniformly loaded beam, } \Delta = \frac{5}{48}\,\frac{f_b\,l^2}{c\,E},\right.$ and $\left.K' = \frac{5}{48\,E}\right)$. Then,

$$K\left(\frac{l}{r}\right)^2 = \frac{\Delta\,c}{r^2} = \frac{K'\,f_b\,l^2\,c}{c\,r^2} = K'\,f_b\,\frac{l^2}{r^2}$$

or $K = K'\,f_b$. That is, the "constant", K, is proportional to f_b. If the eccentricity, Δ, is due largely to initial bends and eccentricities, then K is more nearly a constant, but, in any case, at failure, there will be a considerable degree of bending and hence K will not be a constant, but will increase with an increase in the bending stress, f_b. The fact that K should increase with f_b, or with $\frac{l}{r}$, is indicated in the comparison of the formula with results of tests in the 1926 report of the Committee,[15] in which, the Rankine formula is seen to descend too rapidly from the horizontal for low values of $\frac{l}{r}$. An increasing value of K with proper constants would give a flatter curvature near the origin and larger values of p.

While the Rankine formula, making K a constant, is irrational, it has been widely used, and by selecting proper constants, it can be made to fit results of tests fairly well over the working range. A common method of adjustment is to use a relatively large value of f so as to raise the curve near the Y-axis and then to slice off the top by adopting a maximum limit for p somewhat less than f.

33.—Empirical Formulas.—Many straight-line formulas have been, and still are, in use. These are usually determined with reference to the yield point strength for low values of $\frac{l}{r}$; to the Euler formula for very high values; and to results of tests between these limits. The Johnson parabolic formula is perhaps the best of the empirical formulas so far as it goes. It is a parabola having its vertex on the Y-axis, and drawn tangent to a suitable Euler curve. It is not applicable beyond its point of tangency, which is at a point the y-ordinate of which equals one-half the ordinate for $x = 0$, or $p = \frac{1}{2} f$.

34.—Shear in Columns.—Two cases need to be considered:

(*a*) The shear in a column bent in single curvature: This will be a maximum at the ends. For the theoretical case of the pivoted-end column with eccentric loads, the shear is, from Equation (9),

$$V = P \tan \theta = P\, e\, q \tan \phi = P\, e \sqrt{\frac{P}{E\, I}} \tan \frac{l}{2} \sqrt{\frac{P}{E\, I}} \ldots\ldots (12)$$

For practical calculations it may be assumed that the column at failure is bent in a parabolic curve with an inclination at the end of $\tan \theta = \frac{4\, e'}{l}$, in which, e' is the total central ordinate. The bending stress is, $f_b = \frac{p\, e'\, c}{r^2}$, or $e' = \frac{f_b\, r^2}{p\, c}$; hence, $\tan \theta = \frac{4\, f_b\, r^2}{p\, c\, l}$, and the shear is,

$$V = P\, \frac{4\, f_b\, r^2}{p\, c\, l} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

At failure the bending stress, f_b, is the stress, $f - p$, from any rational column formula, f being the yield point and p, the ultimate unit load on the column. Equation (13) gives the shear corresponding to the ultimate load, P.

(*b*) The shear in a column of a truss with rigid joints where the joint deformation or twisting is such as to cause the column to bend in an

---

[15] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1485.

S-shaped curve. The moments at the ends are the moments causing secondary stresses in the members. Columns subject to such loads are in effect acted on by direct forces applied eccentrically at each end, but of opposite direction, resulting in an obliquity of the applied load. If the eccentricity of each end equals e, the obliquity is $\frac{2\,e}{l}$, and the shear is,

$$V = P\,\frac{2\,e}{l} \qquad (14)$$

The quantity, e, may be expressed in terms of secondary stress or of ratio of secondary to axial stress. The secondary stress (bending fiber stress) due to an eccentricity, e, is $f_b = p\,\frac{e\,c}{r^2}$ and $\frac{f_b}{p} = \frac{e\,c}{r^2} =$ stress ratio $= \alpha$. Then, $e = \frac{\alpha\,r^2}{c}$, and,

$$V = P\,\frac{2\,\alpha\,r^2}{c\,l} \qquad (15)$$

This will be large only for small values of $\frac{l}{r}$. The values of α will vary greatly, being high for short members and for bending in the plane of the truss. In this plane the value of α may easily reach 0.40 or 0.50, but in a plane perpendicular to the truss, which is generally the plane of the lacing, the secondary stresses due to vertical loads will be small, and a value of 0.20 is probably a sufficiently high estimate. This value gives a shear of $V = P\,\frac{0.4\,r^2}{c\,l}$. Comparing this result with Equation (13) it will be seen that it gives higher values of V whenever $\frac{f_b}{p}$ is less than 0.1; that is, for relatively short columns.

35.—Factor of Safety Method in Column Design.—Whatever formula may be used, the logical process in column design is to select a formula which will as nearly as practicable give values of ultimate strength, or some point which represents the practical ultimate value of the column; then to divide the resulting value of average stress, P/A, by the factor of safety corresponding to that used in other members, the object being to produce a structure in which all parts will be stressed to failure, or to their useful limits, simultaneously as the loads are increased.

Columns, unlike other members, are stressed by an amount (maximum fiber stress) which increases at a faster rate than the applied load, and hence if they should be designed so that under working loads the actual maximum fiber stress is, say, one-half the yield point of the material, an increase of load of 50% would produce more than a 50% increase in fiber stress, and the factor of safety for the column would be considerably less than 2.0. In most column formulas (where the load, p, is expressed as an explicit function of f), the working formula is obtained by applying the factor of safety to the entire expression for p (the numerator in the Rankine formula), but in the case of the secant formula this cannot be done. If the factor of safety be applied to the numerator, f, the result will be a value of p which will produce the assumed fiber stress, f. In this case, ultimate values of p must be tabulated, and the factor of safety applied to them. The same comments may be made with respect to shearing stresses. The logical method is to estimate shearing stresses at or near failure and apply a consistent factor of safety to such values for design purposes.

36.—The Action of a Column in a Truss.—The study of the strength of columns has necessarily been based almost wholly on results of tests in a testing machine, where the end conditions have been made as definite as possible, either flat-ended (corresponding generally to a fixed-end condition), or pin-ended, or some approximation to pivot ends. It has been difficult to arrange end conditions of any kind so as to meet precisely the corresponding theoretical conditions. In the case of flat-end tests much shimming has been required to adjust for inaccurately cut ends or non-parallel testing-machine plates. To simulate pivoted ends requires either knife-edge bearings, suitable for very small columns, or some other frictionless bearing. The pin bearing commonly used is far from frictionless, in fact, a pin-ended column in a testing machine acts at first like a fixed-end column, and if reasonably straight, continues so to act nearly or quite up to the maximum load. For pivoted ends, the roller device used in the tests described in Part I is probably the best practicable arrangement. On account of its very small frictional resistance, it gives a close approximation to the truly pivoted end; and because of its definiteness it would seem the most satisfactory arrangement to use in studies for the correlation of tests with theory and for the relative effects of the various elements of column construction. When this is all done, however, there remains the important task of relating the results of these tests and analyses on pivoted-end columns to the behavior of columns in a structure.

To arrive at some judgment in the matter, it is desirable first to consider the action of a compression member in a truss with riveted joints. Consider, first, the action in the plane of the truss. The effect of loading such a structure is to develop certain bending moments in all the members at a joint, giving rise to what are usually known as secondary stresses. These deformations and stresses are caused by the changes in the length of the various members, and are subject to reasonably close calculation. Some of the members will be bent in opposite directions at the two ends, giving rise to an S-shaped curvature; others will be bent in single curvature. The top chord members in a Pratt truss in all panels except the end ones are generally examples of single curvature. The verticals and end posts are likely to be bent in reversed curves. In any case, the angle of bend at each end is a fairly definite and calculable quantity, depending almost wholly on the axial strains in the various members. So far as this bending affects the strength of the compression member, it is obvious that the worst case is where the bending is of single curvature, thereby forcing the member into a form giving the maximum deflection near the center and thus reducing the strength of the member as a column. This bending action always occurs in a truss, but it is an effect not easily produced in a testing machine. This action may be investigated theoretically as follows.

Assume $A B$ (Fig. 33) to be a column in a truss, subjected to an axial load, P, and moments, M, at each end. These moments are due to the deformation of the truss as a whole and, within certain limits, are proportional to the load on the structure and the load, P.

The bending moment, M, at any point, C, a distance, x, from A, is,

$$M = M_1 + P\,y$$

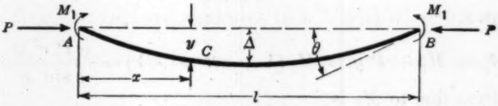


FIG. 33.

Then,

$$\frac{d^2 y}{d x^2} = -\frac{M}{E\,I} = -\frac{M_1}{E\,I} - \frac{P\,y}{E\,I}$$

From this is derived, as in the secant formula:

$$y = \frac{M_1}{P}\left[\sec q\,x - 1\right] \qquad \text{(16)}$$

$$\tan\theta = \frac{q\,M_1}{P}\tan\phi \qquad \text{(17)}$$

and,

$$\text{Center deflection} = \Delta = \frac{M_1}{P}\left[\sec\phi - 1\right] \qquad \text{(18)}$$

Now, θ is the deflection angle fixed by the twist of the joints, A and B, due to the distortion of the truss, and is, up to loads approaching the ultimate, closely proportional to the load or to the stress, P. These angles, θ, are functions from which the secondary stresses may be calculated, and their magnitudes may be estimated by expressing them in terms of the corresponding bending or secondary stresses. For a member bent in single curvature with equal deflection angles, θ, at each end, the corresponding secondary or bending stress at the end is,

$$f_s = \frac{2\,c\,E}{l}\tan\theta$$

Expressing the secondary stress as a fraction, α, of the axial unit stress, p,

$$f_s = \alpha\,p = \frac{2\,c\,E\tan\theta}{l}$$

or,

$$\tan\theta = \frac{\alpha\,p\,l}{2\,c\,E} \qquad \text{(19)}$$

in which, α has a numerical value amounting ordinarily, in the cases considered, to 0.1 to 0.2; and in any particular case $\tan\theta$ will increase directly with the load or the value of p. This value of $\tan\theta$ may then be used in Equation (17); then, solving,

$$M_1 = \frac{\alpha\,p\,l\,P}{2\,c\,E\,q\tan\phi} \qquad \text{(20)}$$

Substituting p_e (Euler's value) for $\dfrac{\pi^2 E}{\left(\dfrac{l}{r}\right)^2}$, and ϕ for $\dfrac{\pi}{2}\sqrt{\dfrac{p}{p_e}}$,

$$M_1 = \frac{\alpha P r^2 \phi}{c \tan \phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

and, at the center,

$$M_c = M_1 + P y = M_1 [1 + \sec \phi - 1] = \frac{\alpha P r^2 \phi}{c \sin \phi}$$

The bending stress due to M_c is,

$$f_b = \frac{M_c c}{I} = \alpha p \frac{\phi}{\sin \phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

This represents the bending stress at the center of the column the ends of which are gradually turned through an angle, θ, proportional to the stress, p, but which are otherwise securely held. If f = total fiber stress (yield point at failure) then, as before,

$$f = p\left(1 + \frac{\alpha \phi}{\sin \phi}\right)$$

and,

$$p = \frac{f}{1 + \dfrac{\alpha \phi}{\sin \phi}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

as the corresponding column formula.

Comparing this with the secant formula for pivoted ends, and assuming $\alpha = \dfrac{e c}{r^2}$, it will be noted that sec ϕ has been replaced by $\dfrac{\phi}{\sin \phi}$. In Fig. 34, Curve A is the secant formula of Equation (10) and Curve B is the formula of Equation (23). In these calculations, f = 36 000, $\dfrac{e c}{r^2} = \alpha = 0.25$, and E = 29 000 000. There is also a plotted curve, C, from Equation (10) for pivoted ends, but using in the formula the half length, corresponding to Euler's formula for fixed ends. This is,

$$p = \frac{f}{1 + \dfrac{e c}{r^2} \sec \dfrac{\phi}{2}}$$

It is noted that Curve B lies only slightly below Curve C, showing that the end conditions assumed are not much different from fixed-end conditions with initial eccentricity due to curvature.

It may be concluded, therefore, that the strength of a column in a truss under the conditions assumed is not much less than that of a pivoted-end column of one-half the length.

In the case of columns bent in a reversed curve by the secondary stress moments, the bending stresses will be a maximum at the ends and small near the center, so that buckling action will be very small. The strength of such columns will be closely determined by the sum of the direct and bending stresses at the ends, as in the case of oblique loading tests given in Part I.

The strength of columns against buckling at right angles to the plane of the truss is not so readily estimated as in the case already considered. The

secondary stress moments and distortions in this direction are certainly quite small in top chord members of through bridges; but, on the other hand, the lateral members do not offer very great resistance to the lateral buckling of chord members in alternate directions in successive panels, giving a much nearer approach to pin-end conditions than the action in the plane of the truss. This apparent relative weakness in a lateral direction can be equalized by increasing the slenderness ratio, $\frac{l}{r}$, in this plane, a variation corresponding to common practice. Vertical posts with floor-beams and sway-bracing attached are thoroughly supported laterally, and act very definitely in the manner assumed in the foregoing analysis. The buckling tendency is small, and the maximum stress is likely to be at the top of the floor-beam. Compression web members in triangular trusses and the like are apt to be much less securely held against lateral bending than against bending in a vertical plane, although when riveted into heavy chord members they are quite well fixed. Here, again, equalization of buckling strength may be given by increasing the value of $\frac{l}{r}$ in a lateral direction.

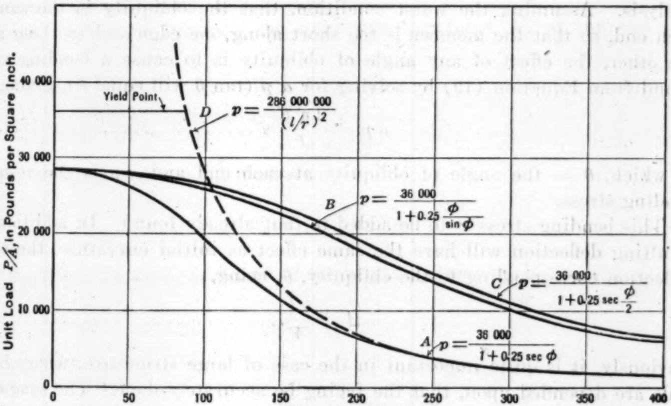


FIG. 34.

37.—Effect of Dead Weight.—The effect of the weight of the member on the center moment is of considerable interest. An exact analysis is quite complex, but an approximate estimate may be made by comparing the center deflection which may occur under the conditions already considered with the center deflection of the column due to its own weight, assuming fixed ends and the column to be horizontal.

The center deflection, not considering weight, is given by Equation (18). Substituting the value of M_1 from Equation (21),

$$\varDelta = \frac{\alpha r^2 \phi\,(1 - \cos \phi)}{c \qquad \sin \phi}$$

The deflection of a beam fixed at the ends is $D = \frac{w\,l^4}{384\,E\,I}$, in which, w = the weight per unit length. This is approximately equal to 0.35Δ lb. per in. of length, whence,

$$D = \frac{0.35\,l^2}{384\,E}\left(\frac{l}{r}\right)^2$$

For purposes of comparison, consider, for example, a slender member with $\phi = \frac{\pi}{2}$; $\frac{l}{r}$ will equal about 108. Assume $l = 648$ in., $r = 6$ in., $c = 8$ in. For these values, $\Delta = 2.16$ in. and $D = 0.15$ in., or only 7% of Δ. Again, take $\phi = \frac{\pi}{4}$, for which $\frac{l}{r}$ is about 75. With the same values of r and c, the deflections will be, respectively, 0.42 in. and 0.032 in., the latter being 7.5% of the former. It appears, therefore, that the effect of weight is of little relative importance under the conditions assumed.

38.—Effect of Obliquity of Flat Ends.—Where dependence is placed mainly on butted ends, with relatively few rivets in splice-plates, the effect of variation from the true angle can readily be determined from the foregoing analysis. Assuming the worst condition, that the obliquity is the same at each end, so that the member is too short along one edge and too long along the other, the effect of any angle of obliquity is to cause a bending stress, found from Equation (19) by solving for $\alpha\,p$ ($\tan\theta$ will equal θ), giving,

$$\alpha\,p = \frac{2\,c\,E}{l}\,\theta \text{ ....} (24)$$

in which, θ = the angle of obliquity at each end and $\alpha\,p$ is the resulting bending stress.

This bending stress is to be added to that already found. In addition, the resulting deflection will have the same effect as initial curvature, the center deflection corresponding to the obliquity, θ, being,

$$\Delta = \frac{\theta\,l}{4} \text{ ....} (25)$$

Obviously, it is quite important in the case of large structures, where butted ends are depended upon, that the facing be accurately done. The magnitude of the allowable obliquity for any assumed value of stress, $\alpha\,p$, is, from Equation (24), $\theta = \frac{\alpha\,p}{2\,c\,E}$. For example, assume $\alpha\,p = 2\,500$ lb. per sq. in. = about 10% of the ultimate strength. Also, let, $\frac{l}{2\,c} = 15$. Then,

$$\theta = \frac{2\,500 \times 15}{30\,000\,000} = 0.00125$$

which is the permissible error in angle of the end face, or an error of 0.030 in. in a width of 24 in.

39.—The Effect of Initial Bends.—In the case here considered, this effect is of the same nature as in the fixed-end column, namely, less than the effect of one-half the same amount of deflection in a pivoted-end column of one-half the length, and can be approximately allowed for by adding to α in the column formula one-half the quantity, $\frac{e' c}{r^2}$, in which, e' = the deflection due to initial bends.

40.—Effect of Deflection Due to Shear.—In the tests of Part I, the relative rigidities of various columns in the plane of the web and at right angles thereto were determined, and it was shown that in the case of the laced channel columns the shearing distortion in the plane of the lacing was considerable under oblique loading. The relative deflections under cross-bending loads were also found, and in the case of the laced columns, the observed deflections in the plane of the lacing were nearly twice as great as the calculated deflections in which the effect of shearing strains was omitted. This raises the question of the effect of shearing distortion on the bending of columns in the plane of lacing as compared to bending in the plane of solid webs. To answer this in part, tests of Series (*C*) were made under eccentric loads, and so far as these tests were concerned, the effect of shearing strain on deflection in the plane of lacing was negligible (see Fig. 29). However, this question is worthy of further analysis.

It will be helpful to compare the shearing and bending stresses in a column with those in a uniformly loaded beam.

From the secant formula, the total shearing force, V, at the end of a column is,

$$V = P\, e q \tan \phi$$

and that per unit of sectional area is,

$$v = p\, e q \tan \phi \qquad (26)$$

The bending stress at the center is,

$$f_b = p \frac{e\, c}{r^2} \sec \phi$$

and, therefore, the shear in terms of the bending stress, f_b, is,

$$v = f_b \frac{q\, r^2}{c} \sin \phi$$

Writing

$$q = \sqrt{\frac{P}{E\, I}} = \frac{\pi}{l} \sqrt{\frac{p}{p_e}}$$

and taking $r = 0.8c$, approximately,

$$v = f_b \frac{2.52 \sqrt{\frac{p}{p_e}}}{\frac{l}{r}} \sin \phi \qquad (27)$$

For a beam uniformly loaded, the bending stress is,

$$f_b = \frac{w\, l^2\, c}{8\, A\, r^2}$$

The end shearing stress per square inch is,

$$v = \frac{w\,l}{2\,A}$$

and hence, with $r = 0.8c$,

$$v = \frac{3.2\,f_b}{\dfrac{l}{r}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Comparing Equations (27) and (28) for the same bending stress, f_b, the ratio,

$$\frac{v\ \text{(column)}}{v\ \text{(beam)}} = \frac{2.52\sqrt{\dfrac{p}{p_e}}}{3.2}\sin\phi = 0.79\sqrt{\frac{p}{p_e}}\sin\phi$$

Calculating this quantity, the values given in Table 27 are found, based on the secant formula for ultimate strength, $p = \dfrac{36\,000}{1 + 0.25\sec\phi}$. The ratio of shear in column to direct stress $\left(\dfrac{V}{P}\right)$ is also given.

TABLE 27.—RELATION OF SHEARS IN COLUMNS AND BEAMS FOR EQUAL BENDING STRESSES.

p.	$\frac{p}{p_e}$.	$\frac{l}{r}$.	$\frac{v\ \text{(column)}}{v\ \text{(beam)}}$, for equal bending stress.	$\frac{V}{P}$.
28 500	0.047	22	0.058	0.0022
28 000	0.1	32	0.12	0.0034
27 100	0.2	45	0.23	0.0052
26 000	0.3	57	0.33	0.0072
24 700	0.4	69	0.41	0.0090
23 000	0.5	79	0.51	0.011
21 000	0.6	91	0.57	0.014
18 200	0.7	105	0.64	0.019
14 500	0.8	125	0.70	0.026
8 900	0.9	170	0.75	0.045

For ordinary values of $\frac{l}{r} = 50$ to 100, the ratio of shears for the same bending stress ranges from about 0.25 to 0.6.

In the cross-bending tests of Part I, the deflection due to shear in the laced channel columns averaged about 0.8 of that due to bending, and the value of $\frac{l}{r}$ was about 20. These bending tests were made with center loads. If they had been made with uniform loads, giving a shear distribution more nearly like that of a column, it can be shown that the average shearing stress for like deflection would have been 0.8 as much and hence the shearing distortions would have been less than those shown. Hence, 0.8 × 0.8, or about $\frac{2}{3}$ may be taken as representing the ratio of deflection due to shear if tested with uniform loads, to that due to bending under a central load. For the columns tested in Series (*C*), $\frac{l}{r} = 21.9$, for which the ratio of column shear to beam shear, as shown in Table 27, is about 0.06, and, hence, the relative shear distortion for this column, as a column, would be about $0.06 \times \frac{2}{3} = 0.04$ of that due to bending. Cross-bending tests of columns with $\frac{l}{r} = 50$, and with similar proportions to those tested, would, in accordance with Equation (28), have shown a relative shear deflection of about $\frac{2}{3} \times \frac{20}{50} = 0.26$; and for a column of $\frac{l}{r} = 50$, the shearing stress for a like bending stress (see Table 27) would be about 0.3 as much as in a beam, and hence the deflection due to shear would be expected to be about $0.3 \times 0.26 = 0.08$ of that due to pure bending as assumed in the column analysis. For $\frac{l}{r} = 100$, the result would be about the same.

This gives an approximate measure of the relative rigidity in the plane of the lacing as compared to that in the plane of the webs. While other proportions of lacing and other tests would doubtless vary these results somewhat, this analysis is sufficient to indicate that shearing distortion is not of great relative effect upon the strength of columns. In this case, the actual effect upon the theoretical ultimate strength could be found by using a value of E in the formula about 10% less than the value for direct stress, or, say, 26 000 000 lb. per sq. in. Any difference in strength in the two directions can be overcome by increasing the value of r, that is, by spreading the channels farther apart.

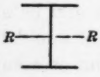
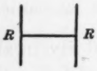
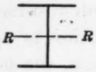
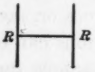
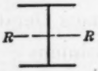
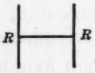
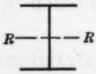
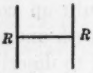
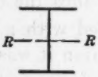
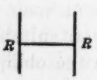
(C) Strength of Columns as Affected by Various Details

1.—Eccentricities in the Wisconsin Test Columns

41.—Method of Measuring Eccentricity and Crookedness.—In order to secure data for determining the gravity axis of each **H**-column, micrometer measurements of both end cross-sections were made and recorded on separate carbon prints. From these data the location of the column axis with respect to the center of the web was calculated. This information was used in centering the columns on the bearing blocks for testing.

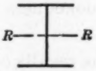
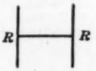
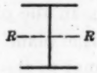
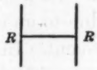
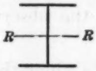
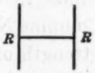
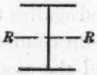
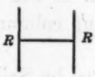
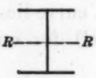
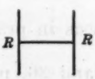
In order to secure some information concerning the straightness of these test columns, two methods of measurement were used. In the more accurate method, which was conveniently used on the short built-up columns, two transits were set up so that the lines of sight described north and south planes about 18 in. apart; and two other transits were arranged to describe similarly spaced east and west planes. Within these imaginary planes the column was set vertically on a base plate which was accurately leveled with a very sensitive spirit-level. By holding steel rules normal to the column it was possible to determine the distance from each of the four imaginary planes to the column. Twelve such measurements, three per plane, were made at each end, at the quarter-points, and at the middle. They were recorded on suitable sketches. Then the column was turned upside down to determine the obliquity of the upper end. From these observations, crookedness, obliquity of ends, and the amount of wind were calculated.

TABLE 28.—ECCENTRICITIES AND CROOKEDNESS OF 8-INCH H-COLUMNS, UNIVERSITY OF WISCONSIN TESTS.

Column No. and length.	Position of rollers.	ECCENTRICITY OF CENTER OF WEB DUE TO ROLLING, IN INCHES.		Crookedness, e′, at mid-section, in inches.	CENTER LOADING TEST ECCENTRICITY AS CALCULATED FROM MEASUREMENTS, IN INCHES.		
		Top.	Bottom.		Top.	Bottom.	Mid-section.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
H 1		+0.02	0.00	No measurements made, but column had good alignment.	0.00	0.00	0.00±
l = 39 in.		+0.02	+0.03		+0.03	+0.04	+0.04±
H 2		0.00	0.00	0.00	0.00	0.00	0.00
l = 99 in.		+0.05	+0.03	+0.12	+0.08	+0.06	−0.05
H 3		+0.02	0.00	+0.02	+0.02	0.00	+0.03
l = 168 in.		+0.04	+0.05	−0.01	−0.03	0.00	−0.02
H 4		+0.02	+0.01	+0.09	−0.07	−0.09	+0.01
l = 238 in.		−0.02	+0.04	+0.03	+0.01	+0.06	+0.07
H 5		+0.01	0.00	−0.01	−0.04	−0.01	−0.04
l = 307 in.		−0.07	+0.01	−0.05 −0.10	+0.04 +0.10	+0.02 +0.08	*−0.02 †−0.01

* Before over-strain. † After straightening.

TABLE 28.—(*Continued.*)

Column No. and length. (1)	Position of rollers. (2)	Eccentricity of Center of Web due to Rolling, in Inches. Top. (3)	Eccentricity of Center of Web due to Rolling, in Inches. Bottom. (4)	Crookedness, e′, at mid-section, in inches. (5)	Center Loading Test Eccentricity as Calculated from Measurements, in Inches. Top. (6)	Center Loading Test Eccentricity as Calculated from Measurements, in Inches. Bottom. (7)	Center Loading Test Eccentricity as Calculated from Measurements, in Inches. Mid-section. (8)
H 6		0.00	−0.01	−0.01	0.00	−0.01	−0.01
l = 40 in.		−0.02	−0.01	+0.01	0.00	0.00	+0.01
H 7		−0.01	−0.03	+0.01	−0.01	−0.03	−0.01
l = 99 in.		−0.01	−0.02	0.00	0.00	+0.01	0.00
H 8		0.00	0.00	+0.02	0.00	0.00	+0.02
l = 168 in.		−0.01	−0.01	0.00	0.00	0.00	0.00
H 9		−0.04	−0.04	+0.08	−0.01	+0.01	+0.08
l = 288 in.		−0.02	−0.04	+0.16	−0.15	−0.16	0.00
H 10		+0.06	0.00	+0.04	0.00	−0.02	+0.03
l = 307 in.		0.00	0.00	+0.23	−0.14	−0.16	+0.08

The straightness of an H-column was determined by supporting it in a horizontal position and making measurements from planes formed by four fine wires. These wires were spaced fixed distances apart and ran along the corners of the column.

The obliquity of the ends of the H-columns was determined by measuring the thickness of the shims required under the lower bearing block to produce parallelism between the top of the column and the upper bearing.

By averaging all measurements in either direction for each section of the column, and comparing these mean measurements with the end means, a measure of crookedness was obtained. Herein, only the crookedness as referred to the mid-section will be reported.

42.—Eccentricities in H-Columns Due to Rolling.—The results of the center of gravity determinations on the 8-in. columns are given in Columns (3) and (4) of Table 28. For these columns the values of $\frac{c}{r^2}$ for the central axis normal to the web is approximately 0.3, whereas for the other gravity axis it is about 1.0. Hence, in a short column, an eccentricity of 0.06 in. in the plane of the web causes a bending stress of only 2%, whereas in the plane normal to the web it amounts to 6 per cent. Such eccentricity normal to the web would correspond to a displacement of the web of about 0.09 in. In a long column with a slenderness ratio of 150, the reduction in strength due to an eccentricity of 0.06 in. normal to the web would amount to about 5 per cent. In most cases the tabulated eccentricities are well below 0.06 in. Hence, aside from inconvenience in making connections to adjoining members, the observed rolling eccentricities are not of importance in these columns.

43.—Crookedness.—The data in Column (5) of Table 28, show that crookedness is a far more important defect than rolling eccentricity in these small H-columns. The bowing in a plane normal to the web in Columns Nos. $H9$ and $H10$ would have caused an appreciable reduction in the strength of these columns had they been tested to destruction with rollers parallel to the webs. In Fig. 35 the crookedness in these H-columns has been plotted against the slenderness ratio. The crookedness at mid-section for the built-up columns has also been plotted against the slenderness ratio in Fig. 36, and the crookedness of both types has been plotted against the length of column in Fig. 37. These data show that, for columns of similar length, the built-up columns were the more crooked.

To enable comparisons to be made, the curves suggested by Salmon[16] have been incorporated in these diagrams. In Fig. 37 Salmon's curve lies well above the field of plotted points. Hence, the crookedness (e') for one of these columns estimated by $e' = \frac{l}{750}$, would in most cases be considerably higher than the actual value. For these tests the crookedness was in practically all cases less than that given by $e' = \frac{l}{1\,500}$. In Figs. 35 and 36, plotted with respect to the slenderness ratio, Salmon's equation, $e' = 0.0023 \frac{l}{r} \left(\frac{l}{435\ r}\right)$, gives values too high for the H-columns and is more nearly a mean line than a limit for the built-up columns. For the H-columns practically all values lie below the boundary line representing $e' = \frac{l}{750\ r}$.

---

[16] "Columns," by E. H. Salmon, Oxford Technical Publications, p. 152.

TABLE 29.—Obliquities in Wisconsin Test Columns.

Column No. (1)	Plane of obliquity referred to web. (2)	Nominal width of column in plane of obliquity, in inches. (3)	Obliquity. In the width of the column, in inches. (4)	Obliquity. Slope, in inches per inch. (5)
H 4	Normal	8	0.005	0.0006
H 6	Normal	8	0.024	0.0030
H 7	Parallel	8	0.003	0.0004
	Normal	8	0.026	0.0033
H 8	Parallel	8	0.026	0.0033
	Normal	8	0.023	0.0029
H 9	Parallel	8	0.024	0.0030
H 10	Parallel	8	0.032	0.0040
	Normal	8	0.045	0.0056
1	Parallel	10½	0.001	0.0001
	Normal	10⁵⁄₁₆	0.033	0.0032
2	Parallel	10	0.015	0.0015
	Normal	10	0.010	0.0010
2 *A*	Parallel	10	0.053	0.0053
	Normal	10	0.008	0.0008
3	Parallel	10½	0.006	0.0006
	Normal	12	0.002	0.0002
3 *A*	Parallel	10½	0.013	0.0012
	Normal	12	0.010	0.0008
4	Parallel	10	0.030	0.0030
	Normal	12	0.002	0.0002
5 *A*	Parallel	10	0.003	0.0003
	Normal	12	0.036	0.0036
6	Parallel	10	0.028	0.0028
	Normal	12	0.004	0.0003
7	Parallel	10	0.006	0.0006
	Normal	12	0.002	0.0002
8	Parallel	10	0.013	0.0013
	Normal	12	0.020	0.0016
9	Parallel	10	0.019	0.0019
	Normal	1	0.007	0.0006

Considering all the results it would seem that the value of e' is more nearly proportional to l than to $\frac{l}{r}$, but the data are too meager to warrant any general conclusion. The relative influence of crookedness in these particular columns can be estimated to some advantage. Assume that $e' = \frac{l}{1\,200}$ and $\frac{c}{r} = 1.2$; then $\frac{e'\,c}{r^2} = \frac{\frac{l}{r}}{1\,000}$. For $\frac{l}{r} = 100$, $\frac{e'\,c}{r^2} = 0.1$, and for $\frac{l}{r} = 150$, $\frac{e'\,c}{r^2} = 0.15$. These values are considerably smaller than those usually assumed in the application of the theoretical formula for eccentric loads.

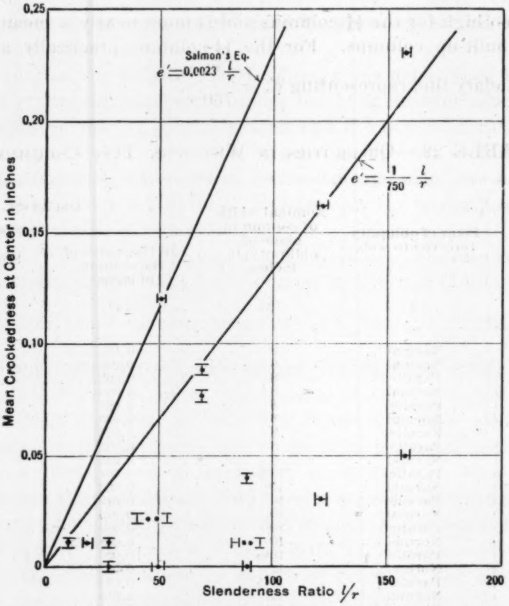


FIG. 35.—RELATION OF CROOKEDNESS TO SLENDERNESS RATIO, H-COLUMNS, UNIVERSITY OF WISCONSIN TESTS.

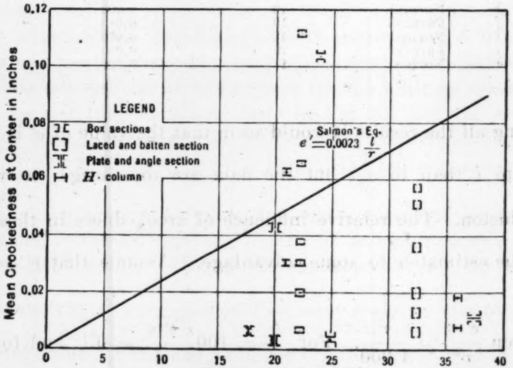


FIG. 36.—RELATION OF CROOKEDNESS TO SLENDERNESS RATIO, UNIVERSITY OF WISCONSIN TESTS.

44.—Eccentricities in Center Loading Tests.—In order to correlate rolling-eccentricity and crookedness measurements of **H**-columns with the position of the respective columns found by trial center loadings, the data in Columns (6), (7), and (8) of Table 28, have been prepared. If the correlation between measurements and trial centering was perfect, all the values in Column (8), Table 28, would be zero. However, most of the eccentricities at mid-section from measurement are small and the data indicate that eccentricity can be predicted with fair accuracy from shape and crookedness measurements.

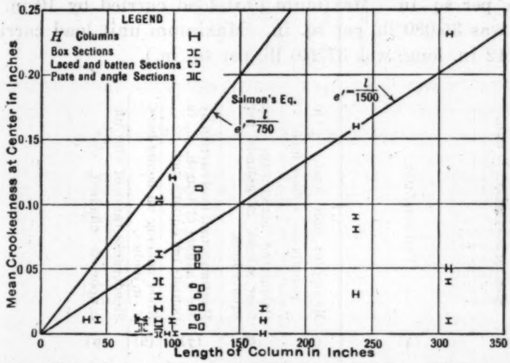


FIG. 37.—RELATION OF CROOKEDNESS TO LENGTH OF COLUMN, UNIVERSITY OF WISCONSIN TESTS.

Correlation between the measured eccentricities and strain measurements was also made by comparing the total eccentricities (measured eccentricities plus corresponding deflection) with the corresponding eccentricities computed by strain measurements.[17] In making this comparison the unit load (P/A) considered ranged from approximately 9 000 lb. per sq. in. for the column of maximum slenderness ratio to 20 000 lb. per sq. in. for some of the short **H**-columns. Considering both built-up and **H**-columns, the comparison of these two sets of eccentricities shows that the maximum discrepancy corresponded to a fiber stress difference of less than 11% and that, on the average, the fiber stress difference due to discrepancies in the values of e would be less than 4 per cent. This close agreement furnishes further evidence of the value of shape and crookedness measurements of columns.

45.—Obliquity of Ends.—In these data, obliquity of ends means simply the difference in slope between two ends of a column. The calculated obliquities for the **H**-columns and the measured values for the built-up columns are given in Table 29. There seems to be no definite relation between the obliquities of the **H**-columns and slenderness ratios or lengths. The average obliquity of these columns is approximately 0.003. In the data for the built-up columns, the obliquity is somewhat greater in the longer columns than in those of shorter length. The data are insufficient, however, to establish definite conclusions regarding such relationship.

---

[17] For the method, see the 1926 report, *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1537.

TABLE 30.—THE EFFECT OF UNSUPPORTED LENGTH OF CHANNEL ON THE STRENGTH OF LATTICED CHANNEL COLUMNS. SINGLE LACING-BARS WITH ALTERNATE SPACING OF RIVETS IN THE FLANGES OF A SINGLE CHANNEL.*

(Maximum unit load carried by 6-in. channel, 6 in. long, was 42 490 lb. per sq. in. Maximum unit load carried by 8-in. channel, 8 in. long, was 43 300 lb. per sq. in. Maximum unit load carried by 10-in. channel, 10 in. long, was 35 080 lb. per sq. in. Maximum unit load carried by 12-in. channel, 12 in. long, was 37 240 lb. per sq. in.)

Column No. (1)	Length, in inches. (2)	Slenderness ratio, $\frac{l}{r}$ min. (3)	End condition. (4)	Maximum unit load, $\frac{P}{A}$, in pounds per square inch. (5)	Calculated maximum unit fiber stress, in pounds per square inch. (6)	Maximum unit load carried by one channel of length, p', in pounds per square inch, (7)	Rivet spacing on lacing-bars, p', in inches. (8)	Slenderness ratio of one unsupported channel, $\frac{p'}{r_c}$. (9)	Kind of failure. (10)
			TWO 6-INCH CHANNELS WITH 8-INCH SEPARATION.						
1059	121	52	Flat	36 700	37 200	36 800	18	38	Buckling
1060	121	52	Flat	35 300	36 000	....	....	....	Buckling
1095	125	58	Flat and Pinned	33 700	33 800	....	....	....	Buckling
1096	125	53	Flat and Pinned	33 800	34 300	....	....	....	Buckling
1107	144	62	Pinned	34 700	39 500	....	....	....	Deflection
1108	144	62	Pinned	34 200	39 100	....	....	....	Deflection
1	150	64	Pinned	35 900	37 900	....	....	....	Deflection
			TWO 8-INCH CHANNELS WITH 8-INCH SEPARATION.						
1111	160	53	Pinned	34 800	35 100	35 300	18	34	Deflection and buckling
1112	160	53	Pinned	35 200	35 600	....	....	....	Deflection and buckling
1113	200	67	Pinned	33 900	34 700	....	....	....	Deflection
1114	200	67	Pinned	33 600	34 600	....	....	....	Deflection
1115	240	80	Pinned	32 600	33 600	....	....	....	Deflection
1116	240	80	Pinned	32 100	35 400	....	....	....	Deflection
1123	280	93	Pinned	32 200	36 400	....	....	....	Deflection
			TWO 10-INCH CHANNELS WITH 8-INCH SEPARATION.						
13	150	42	Pinned	35 600	35 900	34 400	22	33	Buckling
14	150	42	Pinned	35 400	35 900	....	....	....	Buckling
11	200	55	Pinned	33 800	35 700	....	....	....	Buckling
12	200	55	Pinned	34 000	37 000	....	....	....	Buckling
3	250	69	Pinned	33 900	36 600	....	....	....	Buckling
4	250	69	Pinned	33 700	36 700	....	....	....	Deflection
5	300	83	Pinned	34 100	36 800	....	....	....	Deflection and buckling
6	300	83	Pinned	31 900	36 400	....	....	....	Deflection and buckling
26	350	97	Pinned	32 200	35 300	....	....	....	Deflection
27	350	97	Pinned	29 400	33 900	....	....	....	Deflection

* From Report on Tests of Metals and Other Materials, 1881.

TABLE 30.—(*Continued.*)

Column No. (1)	Length, in inches. (2)	Slenderness ratio, $\frac{l}{r}$ min. (3)	End condition. (4)	Maximum unit load, $\frac{P}{A}$, in pounds per square inch. (5)	Calculated maximum unit fiber stress, in pounds per square inch. (6)	Maximum unit load carried by one channel of length, p', in pounds per square inch. (7)	Rivet spacing on lacing-bars, p', in inches. (8)	Slenderness ratio of one unsupported channel, $\frac{p'}{r_c}$. (9)	Kind of failure. (10)
TWO 12-INCH CHANNELS WITH 8-INCH SEPARATION.									
1109	120	27	Pinned	33 400	33 700	36 700	22	26	Buckling
17	120	27	Pinned	35 100	35 100	....	....	....	Buckling
15	180	40	Pinned	33 800	34 000	....	....	....	Buckling
16	180	40	Pinned	35 500	35 800	....	....	....	Buckling
9	240	58	Flat	34 400	34 600	....	....	....	Buckling
10	240	53	Flat	33 600	34 700	....	....	....	Buckling
7	300	67	Flat	32 900	35 100	....	....	....	Buckling
8	300	67	Flat	34 200	35 100	....	....	....	Buckling
28	360	80	Pinned	31 600	34 000	....	....	....	Buckling
29	360	80	Pinned	31 300	35 100	....	....	....	Deflection

Obliquities should be made as small as practicable, especially on short columns, for a difference in the slope of the ends amounting to 0.003 in a column 12 in. wide means a difference in length of the opposite sides of 0.036 in. If such a column, 10 ft. long, rests against rigid end bearings and carries a load sufficient to bring the bearings in full contact (P/A = 4 500 lb. per sq. in., or more), it can readily be shown that the fiber stress caused by the resulting bending of the column will be 4 500 lb. per sq. in.

2.—Unsupported Length and Width of Column Components

46.—Unsupported Length of Channels in Latticed Channel Columns.—Table 30 shows data from tests of wrought-iron latticed channel columns reported in 1881.[18] The maximum fiber stresses in Column (6) Table 30, were calculated from the reported deflections on the assumption of fixed ends. Column (7) shows failure loads, P/A, for single flat-end channels having the same length as the unsupported channels in the corresponding fabricated columns. Column (9) shows the ratio of unsupported length of channel to least radius of gyration for a single channel. By comparing the results in Columns (9) and (10) it will be observed that the last set of 12-in. channel columns had the lowest ratio of $\frac{p'}{r_c}$ (= 26) and that a buckling failure occurred in tests of all these columns. The maximum unit fiber stresses for these columns in Column (6), Table 30, were in every instance high and it is probable that the support from the lacing-bars was sufficient in all cases to develop the yield point of the channels before buckling took place.

---

[18] Report on Tests of Metals and Other Materials, Watertown Arsenal, 1881.

Bouscaren's[19] latticed channel wrought-iron Columns Nos. 37 and 38 failed by buckling of the channels between the points of attachment of the lacing-bars (single lacing). The maximum unit loads were 29 600 and 32 300 lb. per sq. in., respectively. The ratio, $\frac{p'}{r_c}$, for these columns was 28. Bouscaren's Column No. 41, with $\frac{p'}{r_c} = 25$, approximately, and other dimensions similar to Column No. 37, failed by buckling and bending as a whole under a unit load of 32 400 lb. per sq. in.

Talbot and Moore[20] report failure of their steel Column No. 1 by buckling of the flanges between the lacing-bars at $P/A = 23\,450$ lb. per sq. in. The ratio, $\frac{p'}{r_c}$, for that column was 38.

The two large double latticed columns,[21] LC_0LC_1, made with four ribs, failed by buckling of the ribs between lattice-bars. For the column of carbon steel, $\frac{p'}{r_c}$ was about 27; for the Mayari steel column, $\frac{p'}{r_c}$ was approximately 23. Both these columns failed at unit loads below the yield point of the metal. These failures were undoubtedly influenced by the character and arrangement of the lattice.

Of the American Railway Engineering Association latticed channel columns, Types *A* to *E* (1915 Series) inclusive, none failed by buckling of the channels between lacing-bars. The slenderness ratios for these columns varied from 50 to 120, but the largest ratio, $\frac{p'}{r_c}$, for a single channel was only 16.

Whereas, from these fragmentary data no accurate conclusion can be drawn concerning the proper limiting value of $\frac{p'}{r_c}$ in latticed channel columns, the data indicate that no failures primarily due to buckling occurred for values of $\frac{p'}{r_c}$ less than 25. Furthermore, in none of the cases under consideration, where the ratio, $\frac{p'}{r_c}$, was greater than 26, is there any evidence of marked reduction in strength which can be directly attributed to local buckling of the channels.

47.—Unsupported Length of Channels in Columns Tied with Batten-Plates.—Table 31 shows the principal results of the American Railway Engineering Association tests of 1918 on channel columns tied with widely spaced battens. Unfortunately, this series of tests did not include columns with battens spaced closely enough to develop the full strength of the column. By plotting the ratio, $\frac{A}{B}$, in Column (7), Table 31, against the slenderness ratio for a single channel (Column (3)), it appears that if the battens had been spaced so that the ratio of the unsupported length to least radius of gyration for a single channel $\left(\frac{p'}{r_c}\right)$ was 28, the full strength of the column would have been secured. However, since the unsupported channels of batten-plate columns are subjected to heavy bending stresses[22] when the load becomes slightly oblique to the channel webs, the proper ratio for $\frac{p'}{r_c}$ for such channels should be the objective of further tests.

---

[19] *Transactions*, Am. Soc. C. E., Vol. IX (1880), p. 447.

[20] *Bulletin No. 44*, Univ. of Illinois Eng. Experiment Station; also, *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 202.

[21] *Technologic Paper No. 101*, U. S. Bureau of Standards.

TABLE 31.—EFFECT OF UNSUPPORTED LENGTH OF CHANNELS ON CENTRALLY LOADED COLUMNS OF AMERICAN RAILWAY ENGINEERING ASSOCIATION SERIES.

Column No.	SLENDERNESS RATIOS.			Average maximum unit load for three columns, in pounds per square inch, A.	Average maximum unit load of adequately laced columns of same length and sectional area, in pounds per square inch, B.	Ratio. $\frac{A}{B}$
	Whole column, $\frac{l}{r}$.	Single channel, $\frac{p'}{r_c}$.	Ratio, $\frac{\frac{l}{r}}{\frac{p'}{r_c}}$.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1 *K*	85	66	0.77	26 900	34 000	0.79
1 *I*	85	84	0.99	25 400	34 000	0.75
1 *L*	85	81	0.95	24 100	34 300	0.70
1 *M*	120	121	1.01	15 100	30 800	0.49

48.—Unsupported Length in Segmental Columns.—In Bouscaren's tests on Keystone columns[23] failure occurred by bulging of the segments between the rivets when the unsupported length of the segment was 15 in. No bulging took place when the unsupported length was reduced to 12 in. For the latter length the ratio, $\frac{p'}{r_c}$, for the segment was 27.

Bouscaren's tests also indicate that the strengths of his Keystone columns (metal, $\frac{1}{4}$ to $\frac{5}{16}$ in. thick) were improved by reducing the pitch of the rivets to 3 in. near the ends of the columns (see Columns Nos. 3, 8, 25, and 31).

49.—Unsupported Width of Plates in Columns.—Aside from the early work of Hodgkinson[24] and Fairbain,[25] and the tests of the U. S. Bureau of Standards made in connection with the building of the Delaware River Bridge,[26] there is little experimental data relating to the effect of width of unsupported plate on its resistance to axial compression.

In Column (10) of Table 10[27] of the Progress Report of 1926 is recorded the ratio,

$$\frac{\text{Unsupported width of plate between rivet lines}}{\text{Thickness of plate}}$$

---

[22] See "Stresses in Channels of Batten-Plate Columns," Table 12.

[23] *Transactions*, Am. Soc. C. E., Vol. IX (1880), p. 447.

[24] "The Britannia and Conway Tubular Bridge," by E. Clark, Lond., 1850; also the Report of Commission Appointed to Inquire into Application of Iron Railway Structures, *Parliamentary Paper 29*, Lond., 1849.

[25] *Philosophical Transactions*, Vol. 140, pp. 677–725, 1850.

[26] *Technologic Paper No. 327*, U. S. Bureau of Standards.

[27] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), pp. 1546–1547.

for the box-sections, consisting of two channels and two cover-plates reported in Tests of Metals and Other Materials (1884). From these data it will be noted that the short wrought-iron columns with 8-in. channels and 12½ by $\frac{5}{16}$-in. plates, had ratios of unsupported width to thickness ranging from 35 to 40.4. The maximum unit loads, P/A, for these columns varied between 32 100 and 35 000 lb. per sq. in., the maximum fiber unit stress calculated from loads and deflections varied between 35 100 and 37 600 lb. per sq. in., and all columns save Column No. 379 failed by deflection. Column No. 379, having plates with a ratio of unsupported length to thickness equal to 23 and a ratio of unsupported width to thickness equal to 40.4, failed by buckling of plates at a unit load of 35 000 lb. per sq. in., which probably is fully as high as the yield point of the material.

The evidence from these column tests indicates that the ratios of unsupported width of plate to thickness of 40 or less caused no lessening of the axial resistance to compression of these wrought-iron plates.

In connection with the design of the towers for the Delaware River Suspension Bridge, tests were made by R. S. Johnston,[28] of the U. S. Bureau of Standards, on fourteen I-columns, 10 ft. long, consisting of web-plates, 35 in. wide, stiffened on the long edges by four 6 by 4 by ⅜-in. flange angles attached with 4-in. legs outstanding. The web-plate thickness ranged from ⅜ in. to 1¼ in. in thickness. In four of the thicker webs two plates were stitch-riveted together. The ends of the columns were stiffened by 4 by 4 by ½-in. angles and filler-plates, and were carefully milled to afford true bearings for the load. Longitudinal deformations were taken on four gauge lines located on the flange angles. The lateral buckling of the web-plates was measured at nine points across each member (four points on the angles and five on the web-plate) on lines spaced 3 in. apart along the member.

Table 32 gives the principal results of these tests. Measurements of the initial contours of these plates show that there were pronounced initial waves or buckles in the web-plates, ⅜ in. and less in thickness. The largest waves were noted in Webs *G1A*-3, *G2*-4, and *G3A*-3. In Web *G3A*-3 waves having crests ⅛ in. on either side of the axial plane were observed. The contours of the thicker web-plates were more nearly plane. In the tests of these thin web members the presence of these initial waves (crookedness) contributed to premature buckling.

Another noteworthy influence which caused some variation in these tests was the edge restraint produced by the 4 by 6-in. angles. The load buckle curves indicate that this restraint was sufficient to hold the thin, but not the thick, plates in line. Therefore, in Table 32 the ratio of width to thickness of plate has been recorded both for a width of 26 in. between inner rivet lines (the usual specifications stipulation) and for a width of 35 in., which was the full width of these members.

In Fig. 38 the ratios,

$$\frac{\text{Proportional limit from the load buckle curve of the column}}{\text{Proportional limit of the web-plate material}}$$

---

[28] *Technologic Paper No. 327,* U. S. Bureau of Standards.

TABLE 32.—COMPARISON OF PROPORTIONAL LIMITS OF COLUMNS AS DETERMINED FROM LOAD BUCKLE CURVES WITH PROPORTIONAL LIMIT OF MATERIAL, DELAWARE RIVER BRIDGE TESTS.

Column No.	Web thickness, in inches.	UNIT STRESSES, IN POUNDS PER SQUARE INCH, AT PROPORTIONAL LIMIT OF:		Ratio, $\frac{C}{M}$.	RATIO OF UNSUPPORTED WIDTH TO THICKNESS OF PLATE, $\frac{w}{t}$, CONSIDERING THE WIDTH, IN INCHES.	
		Load buckle curve for column, *C*.	Web material, *M*.		26	35
(1)	(2)	(3)	(4)	(5)	(6)	(7)
G 1	$\frac{3}{8}$	20 000	47 000	0.43	69	93
G 1 *A*-3	$\frac{3}{8}$	18 000	32 500	0.55	....	....
G 2-4	$\frac{1}{2}$	21 000	36 800	0.57	52	70
G 2 *A*-4	$\frac{1}{2}$	25 500	42 000	0.61	....	....
G 3-1	$\frac{5}{8}$	35 000	37 000	0.95	42	56
G 3 *A*-3	$\frac{5}{8}$	27 000	32 000	0.84	....	....
G 4-4	$\frac{3}{4}$	40 000	39 500	1.01	35	47
G 4 *A*-4	$\frac{3}{4}$	38 000	38 000	1.00	....	....
G 5-1	1	36 000	34 000	1.06	26	35
G 5 *A*-1	1	34 000	31 500	1.08	....	....
G 6-1	2 by $\frac{1}{2}$ = 1	36 000	36 800	1.08	26	35
G 6 *A*-1	2 by $\frac{1}{2}$ = 1	38 000	42 000	0.91	....	....
G 7-1	2 by $\frac{3}{4}$ = $1\frac{1}{2}$	37 000	39 500	0.94	17	23
G 7 *A*-1	2 by $\frac{3}{4}$ = $1\frac{1}{2}$	41 000	38 000	1.08	....	....

found in Column (5) Table 32, have been plotted against the ratio,

$$\frac{\text{Width of the member } (w)}{\text{Thickness of the web-plate } (t)}$$

Fig. 38 shows that in most cases the proportional limit of members having webs ¾ in. or more in thickness, was reached prior to buckling. The record indicates that variability in material and crookedness were influential in creating the low buckling values of Webs *G6A*-1 and *G7*-1. Web *G3*-1, a fairly plane ⅝-in. web-plate, carried 95% of the proportional limit of the material before buckling became pronounced.

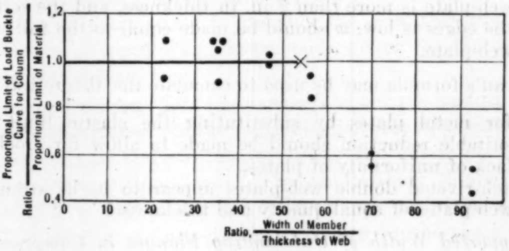


FIG. 38.

A formula for the calculation of the unit load, *p*, at which lateral buckling will occur in an axially loaded plate obtained from Bryan's discussion[29] on stability of ship plating under axial thrusts, is,

$$p = \frac{\pi^2 E t^2}{3 (1 - m^2) w^2} \dots\dots\dots\dots\dots\dots\dots\dots (29)$$

---

[29] *Proceedings*, Mathematical Soc., Vol. 22, p. 54, 1890.

If $m = 0.25$ and $E = 30\,000\,000$ lb. per sq. in., this may be changed to,

$$\frac{w}{t} = \frac{10\,300}{p^{\frac{1}{2}}} \qquad (30)$$

In the tests by Johnston the average proportional limit of the web-plate material was 37 700 lb. per sq. in. If that value be substituted in the equation for p, $\frac{w}{t}$ is found to be 53. This limiting value is indicated by the cross in Fig. 38. For this ratio, elastic failure would occur simultaneously due to compression and buckling. Considering w equal to the full width of the member, this limiting ratio of $\frac{w}{t}$ based on a perfect plate and ideal supporting conditions is slightly less than $\frac{w}{t}$ for the ⅝-in. plates and about 13% more than $\frac{w}{t}$ for the ¾-in. plates. From the existing data, it appears:

1.—If a plate is subjected to thrusts in the direction of its longitudinal axis and restrained along its long edges, it may fail in either of two ways: (*a*) If the width of the plate is not too great it will fail by direct compression, and failure will commence when the fiber stress reaches the elastic limit of the material; (*b*) if the width is large the plate will fail by lateral buckling before the unit load reaches the elastic limit of the material in compression.

2.—The maximum ratio of width to thickness, $\frac{w}{t}$, which a structural steel plate may have without reduction of its compression strength has been conservatively stated by Fairbain to be 30. The more recent tests indicate that for web-plates the limit of $\frac{w}{t}$ may tentatively be set at 50, if w is taken as the full width of the member; and at 40, if w is taken as the distance between rivet lines. If the stiffness of the edge supports is high, w should be taken equal to the distance between the rivet lines. When the web-plate is more than ⅝ in. in thickness, and the restraint along the edges is low, w should be made equal to the full width of the web-plate.

3.—Bryan's formula may be used to calculate the theoretical limit of $\frac{w}{t}$, for metal plates by substituting the elastic limit for p, but suitable reduction should be made to allow for crookedness and lack of uniformity of plates.

4.—Stitch-riveted double web-plates appear to be as strong as single web-plates of equal quality and thickness.

50.—Unsupported Width of Outstanding Flanges in Compression.—From analysis supplemented by tests, Roark[30] has shown that if a flange of uniform thickness, t, and perfectly restrained along one edge, is subjected to uniform compressive fiber stress, the unsupported edge will buckle laterally when the unit load, p, exceeds,

$$p = 0.6\,E\left(\frac{t}{w'}\right)^2 \qquad (31)$$

---

[30] *Engineering Record*, Vol. 68, p. 722; Vol. 74, p. 592.

in which, w' is the unsupported width of the outstanding flange. If p be made the proportional limit of the material, the ratio, $\frac{w'}{t}$, at which failure due to direct compression and lateral buckling would simultaneously occur, may be found. Using a proportional limit of 30 000 lb. per sq. in. for structural steel, $\frac{w'}{t} = 24$. Hence, allowing for imperfections in such material, if the unsupported width of the outstanding flange is less than about 20, the flange should develop full compressive strength before buckling.

3.—Strength and Stiffness of Lateral Connections

51.—Lateral Connections.—The efficiency of lateral connections between the longitudinal components of a column is one of the most important elements of column design, and one of the most difficult to determine either from analysis or experimentation. Assuming integral action, the tests and analysis heretofore given indicate that the strength of a column can be closely calculated if the end conditions and the yield point of the material are known. Integral action of built-up columns depends, however, on the effectiveness of lateral connections, and this is a function not only of the strength of such connections, but also of their stiffness or rigidity. The theoretical analysis given in Section (D) shows that there are two quite different conditions under which the shearing stress to be carried by lateral connections may be large: (*a*) Relatively long columns bending in single curvature; and (*b*) relatively short columns which are subjected to heavy shears by the bending moments developed at the joints, either from the general distortion of the structure, or by unequal or oblique bearings. Columns with truly centrally applied loads are subjected to only small shears, even when the initial bending may be considerable, but a truly central load is a condition hardly to be realized in the case of a column forming a member of a truss. The worst case of shear stress is probably the relatively short column subjected to moments of opposite sign at two ends, giving the effect of an obliquely applied compression load.

52.—Laced and Batten-Plate Columns.—The oblique loading tests described in Part I were undertaken for the particular purpose of securing information regarding the stiffness and strength of lacing and batten-plates of relatively short columns under oblique loads. The results there obtained under shearing stresses of 2 to 5% of the direct stress are useful as bearing on this problem. The relative stiffness is clearly shown by the shearing distortions. The batten-plate columns were naturally the most flexible and the weakest, but the shearing stress of 5% is very high, and is not likely to occur except in the case of very short columns or under abnormal conditions. For more normal values, such as would occur in the ordinary case, the batten-plate column may be a satisfactory and economical design. More study and experimentation are needed on this subject.

The laced channel columns under oblique loading showed no unexpected behavior. The very thin lacing failed at a value corresponding to its strength separately determined. Otherwise, the full strength of the channels was developed.

53.—Strain Measurements in Lacing-Bars.—Careful strain measurements were made on the lacing-bars of the columns described in Part I under both central and oblique loading, and similar measurements have also been made by other investigators. Such measurements give valuable information regarding stresses due to eccentricities and other causes not subject to calculation, and which have a bearing on practical design.

Although there is no shear stress in an ideal column axially loaded, various defects in the column and method of loading cause such stress in the centrally loaded column of practice. Information regarding the importance of such stresses in laced channel columns is afforded from the tests of Talbot and Moore,[31] from the American Railway Engineering Association series,[32] and from the tests at the University of Wisconsin.

All the columns herein considered had single lacing save the Talbot and Moore Column No. 2(*a*), which had double lacing riveted at mid-length. The columns tested by Talbot and Moore had pin ends with pin axes parallel to the lacing; those of the American Railway Engineering Association series had square ends; and those tested at the University of Wisconsin were loaded through roller bearings with the axes of the rollers normal to the lacing. Additional data on the properties of the Wisconsin columns may be found in Table 1 and Fig. 4. From the strain measurements, considering E to be 29 000 000 lb. per sq. in., the unit stress, S_b, in the most stressed bar of each of thirty different columns, has been calculated and recorded in Column (8) of Table 33. The corresponding excess shears, V', found in Column (9), Table 33, were obtained from the relation,

$$V' = S_b \, A_b \sin \theta_b \dots\dots\dots\dots\dots\dots\dots (32)$$

in which, A_b is the cross-sectional area of the lacing-bars in a given panel, or a column, and θ_b is the inclination of the bar to the column axis. The ratios of these calculated shears to the load on the column, $\frac{V'}{P}$, are shown in Column (10).

Fig. 39, compiled from the data of Table 33, indicates that the lacing-bars of short columns are likely to be more severely stressed than the bars of long columns. This is undoubtedly due to inequalities in end bearings as heretofore mentioned, which would cause greater shear stresses in short, than in long, columns.

Further evidence of shear stresses in excess of the nominal amount is afforded by the results of Table 34. These data were obtained from the oblique loading tests made at Wisconsin. In a similar manner, the shears, V_1, corresponding to the most stressed bar in each column, were calculated and recorded in Column (7), Table 34. Then the excess shears, V', and the ratios, $\frac{V'}{P}$, were entered in Columns (10) and (11), Table 34.

---

[31] *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 202.

[32] *Proceedings*, Am. Ry. Eng. Assoc., Vol. 16, p. 636; Vol. 19, p. 789.

TABLE 33.—MAXIMUM UNIT STRESSES IN LACING-BARS CALCULATED FROM MEASURED STRAINS AND CORRESPONDING TRANSVERSE SHEARS FOR AXIALLY LOADED COLUMNS.

Column No.	Series of tests.	Area of column, in square inches.	$\frac{l}{r}$ for column	LACING-BARS.		Unit load on column, $\frac{P}{A}$, in pounds per square inch.	Maximum unit stress, S_b, in lacing-bars,* in pounds per square inch.	Excess shear, V', in pounds.	Ratio, $\frac{V'}{P}$.
				Number measured.	Nominal size, in inches.				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	University of Illinois.	18.76	37	52	1 by ¼	10 000	+ 8 200	8 700	0.020
1	"	18.76	37	36	1¼ by 7/16	10 000	+ 3 000	2 900	0.016
2*a*	"	17.64	38	32	2½ by ⅜	10 000	+ 3 900	5 200	0.029
17	A. R. E. A.	9.56	85	6	2¼ by ⅜	15 000	+10 500	15 080	0.105
58	"	9.56	85	4	2¼ by ⅜	31 000	+23 900	34 300	0.116
58	"	9.56	85	4	2¼ by ⅜	15 000	− 1 450	2 080	0.014
61	"	9.56	85	6	2¼ by ⅜	16 000	+ 1 090	1 560	0.010
66	"	9.56	120	6	2¼ by ⅜	31 000	+34 100	48 900	0.165
66	"	9.56	120	6	2¼ by ⅜	28 000	− 1 810	2 600	0.010
74	"	9.56	50	6	2¼ by ⅜	16 000	− 1 450	2 080	0.014
191	"	9.56	85	6	2¼ by ⅜	30 000	+ 2 540	3 640	0.013
192	"	9.56	85	6	2¼ by ⅜	17 000	+ 1 450	2 080	0.015
197	"	9.56	85	6	2¼ by ⅜	16 000	− 1 090	1 560	0.010
25	"	12.5	120	5	2¼ by ⅜	17 000	− 1 810	2 600	0.012
25	"	12.5	120	5	2¼ by ⅜	30 000	+ 2 900	4 160	0.011
60	"	12.5	85	6	2¼ by ⅜	16 000	+ 1 450	2 080	0.010
62	"	12.5	85	6	2¼ by ⅜	16 000	+ 1 450	2 080	0.010
67	"	12.5	120	6	2¼ by ⅜	15 000	− 1 090	1 560	0.008
68	"	12.5	120	6	2¼ by ⅜	28 000	+ 2 540	3 640	0.010
72	"	12.5	50	6	2¼ by ⅜	16 000	− 1 090	1 560	0.008
73	"	12.5	50	6	2¼ by ⅜	16 000	+ 1 450	2 080	0.010
73	"	12.5	50	6	2¼ by ⅜	31 000	+ 3 620	5 190	0.012
172	"	9.56	120	6	2 by ¼	15 000	+ 725	615	0.004
173	"	9.56	120	6	2 by ¼	32 000	+ 4 350	3 700	0.012
174	"	9.56	120	6	2 by ¼	21 000	− 2 900	2 460	0.012
174	"	9.56	120	6	2 by ¼	33 000	+ 5 800	4 930	0.016
199	"	9.56	85	5	2 by ¼	5 000	+ 725	615	0.013
200	"	9.56	85	5	2 by ¼	15 000	+ 725	615	0.004
201	"	9.56	85	5	2 by ¼	5 000	+ 725	615	0.013
201	"	12.5	85	6	2 by ¼	15 000	− 3 620	3 080	0.016
201	"	12.5	85	6	2 by ¼	19 000	+ 1 450	1 230	0.005
202	"	12.5	85	6	2 by ¼	15 000	+ 2 900	2 460	0.013
202	"	12.5	85	6	2 by ¼	24 000	+ 4 350	3 700	0.012
203	"	12.5	85	6	2 by ¼	15 000	+ 725	615	0.003
4	University of Wisconsin	11.52	22	32	2¼ by 3/16	14 000	+ 5 070	3 780	0.023
5	"	11.30	22	32	2½ by ¼	14 000	− 1 940	2 080	0.013
6	"	11.30	22	33	2½ by ⅜	14 000	− 3 380†	5 650	0.036
	"	11.30	22	33	2½ by ⅜	14 000	+ 2 560	4 270	0.027

* Calculated from strain measurements made only on the outer faces of the bars except in the Illinois tests where they were made on opposite faces. Tension, +; compression, −.

† Bar had an initial bow at middle of 0.01 in. outward.

From a consideration of the $\frac{V'}{P}$ ratios (Table 33), for loads less than the maximum, it will be observed that only 5 columns out of 30 tested exhibited strains corresponding to values of V', larger than $0.02P$. For loads less than the maximum, only 2 of the 335 bars under observation exhibited strains corresponding to values of V' in excess of $0.03P$. The high value, $V' = 0.105P$, for Column No. 17, at a unit load of 15 000 lb. per sq. in., appears abnormal. This conclusion is substantiated by the fact that the strain in the corresponding bar diminished considerably when the column was subjected to higher loads. In one bar of Column No. 58 and one of Column No. 66, high strains were measured at loads near the maximum, but there is no evidence recorded that weakness in these bars caused failure.

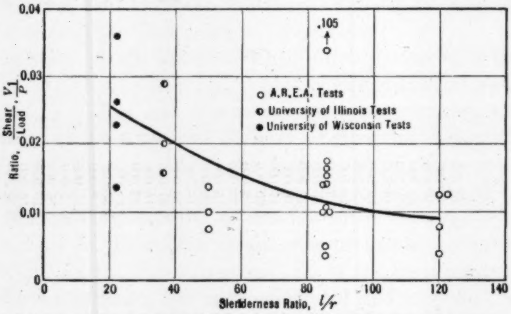


FIG. 39.—RATIO OF SHEAR TO LOAD CALCULATED FROM MOST STRAINED LACING-BAR IN CENTRALLY LOADED COLUMNS.

Only one failure, that reported by Talbot and Moore of their Column No. 1, when equipped with 1 by ¼-in. lacing-bars under a "slightly" oblique load of 150 000 lb. (8 060 lb. per sq. in.), is attributed solely to weakness in the lacing-bars. Since these flimsy bars when separately tested failed under compressive loads of 2 100 lb., Talbot and Moore calculated the equivalent shear in the column at bar failure to have been $V' = 0.025P$.

The data in Columns (6) and (11) of Table 34 show that in most cases where the calculated stresses and excess shears were high, the bars had initial bends which would contribute an increase to the strain measurements. Since no measurements of crookedness in the lacing were reported in the University of Illinois or American Railway Engineering Association tests, no estimate of the effect on measured strains can be made. It appears likely, however, from the results of Table 34, that the high values recorded in Table 33 for certain American Railway Engineering Association columns were enlarged by crookedness of the bars.

Referring again to the results of the Wisconsin tests on obliquely loaded laced columns, it will be recalled that the maximum stresses in excess of the calculated ones were found in tension bars (Table 9). Furthermore, it will be recalled that in only the abnormally thick bars of Column No. 6, carrying small strains which were difficult to measure accurately, did the compression bars exhibit stresses much in excess of the load stresses.

TABLE 34.—MAXIMUM STRESSES IN LACING-BARS CALCULATED FROM MEASURED STRAINS AND CORRESPONDING SHEARS IN EXCESS OF NOMINAL SHEARS DUE TO OBLIQUE LOADING.

Column No.	Series of tests.	LACING-BARS.		Unit load on column, $\frac{P}{A}$, in pounds per square inch.	Maximum unit stress, S_b, in bars,* in pounds per square inch.	Corresponding shear, V_1, in pounds.	Obliquity of load.	Nominal shear, V, in pounds.	Excess shear, $V' = V_1 - V$, in pounds.	Ratio, $\frac{V'}{P}$.
		Number tested.	Nominal size, in inches.							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	University of Illinois	21	1¼ by 7/16	10 000	— 4 900	4 800	0.016	3 000	1 800	0.009
2a	" " "	32	2½ by ⅜	10 000	+ 4 350	5 800	0.0057	1 000	4 800	0.027
	" " "	32	2½ by ⅜	10 000	+ 4 350	5 800	0.0114	2 000	3 800	0.021
4	University of Wisconsin	33	2¼ by 3/16	14 000	+ 8 700†	6 480	0.02	3 230	3 250	0.020
	" " "	33	2¼ by 3/16	14 000	+ 6 760	5 040	0.02	3 230	1 810	0.011
5	" " "	32	2½ by ¼	14 000	+ 8 220‡	8 830	0.02	3 160	5 670	0.036
	" " "	32	2½ by ¼	14 000	+ 4 830	5 190	0.02	3 160	2 030	0.013
5a	" " "	28	2¼ by ¼	14 600	— 5 110‡	5 150	0.02	3 390	1 760	0.010
6	" " "	33	2½ by ⅜	14 000	— 2 900	4 850	0.02	3 160	1 690	0.011
4	" " "	28	2¼ by 3/16	14 600	+15 130	11 290	0.05	8 390	2 900	0.017
	" " "	28	2¼ by 3/16	17 600	+16 270	12 110	0.05	10 110	2 000	0.010
5	" " "	32	2½ by ¼	14 000	+15 000§	16 100	0.05	7 910	8 190	0.052
	" " "	32	2½ by ¼	14 000	+11 600†	12 500	0.05	7 910	4 590	0.029
	" " "	32	2½ by ¼	14 000	— 9 670	10 400	0.05	7 910	2 490	0.016
5a	" " "	28	2¼ by ¼	14 600	— 9 060	9 130	0.05	8 465	665	0.004
	" " "	28	2¼ by ¼	19 600	+13 250‖	13 350	0.05	11 370	1 980	0.009
6	" " "	28	2½ by ⅜	14 600	+ 8 150	13 600	0.05	8 225	5 375	0.033
	" " "	28	2½ by ⅜	19 600	— 8 850	14 800	0.05	11 050	3 750	0.017

* Strains measured along two faces of bars except Columns Nos. 4 and 6 tested with obliquity of 0.02 and Column No. 5; tension, +; compression, —.

† Bar had an initial bow at middle, 0.14 in. inward.

‡ " " " " " " " 0.01 in. "

§ " " " " " " " 0.07 in. "

‖ " " " " " " " 0.01 in. outward.

These results indicate that under large definitely known shears the stresses in the lacing-bars tend to approach the calculated values under the higher loads, but in design the excess stresses shown in Table 34, Column (11), indicate that the obliquity should be liberally estimated. Under nominally central loads the stresses in the lacing-bars will be considerable and larger for the shorter columns. The data of Table 33, Column (10), indicate a shearing stress generally of $V' = 0.015P$ to $0.02P$, with larger values for the shorter lengths.

In order to secure a rough estimate of the magnitude of the stress differences in channels of centrally loaded laced columns, and thus determine the stress transfer which must be made by the lacing, the average fiber stresses near the stay-plates of Columns Nos. 4, 5, 5*A*, and 6 were calculated from strain measurements over the adjoining gauges. They show, for unit loads, P/A, of approximately 15 000 lb. per sq. in., that this stress difference was usually less than 10%; in Column No. 6, the worst case, it reached 20 per cent.

54.—Strain Measurements in Lattice Systems of Large Bridge Columns.—In the tests of large bridge columns reported in *Technologic Paper No. 101*, U. S. Bureau of Standards, strain measurements were made on the lattice members of several of the columns. The results of some of these measurements are given in Table 35, in terms of the stress per square inch. All the columns listed had longitudinal diaphragms except the last two. All had transverse diaphragms. The columns of the first three types were provided with double lacing set at 45° with the column axis. Columns of the first type had no transverse bars. Columns of the second and third types had transverse angles at each panel point which restrained the lateral spreading of the outer ribs. Column US_2MS_1 (carbon steel) had four ribs and was provided with a cover-plate on one side. On the opposite side the outer flanges were tied with a single system of lacing-bars. These had independent ends, and served to connect each outer rib with the adjacent inner rib. Bars were set at an angle of 60° with the column axis. The inner ribs were connected by the tie-plates placed at each intersection of the lattice-bars. Column LC_0LC_1 (carbon steel) had four ribs which were tied on both flanges in a manner similar to Column US_2MS_1.

Considering the first type, without transverse bars, there is little resistance, except near the batten-plates, to the outward bending of the flanges which results from the pantograph action of the latticing when the column is under compressive stress. The result is that the lattice-bars show very little stress (1 500 to 2 000 lb. per sq. in. as a maximum), except in the panel next to the batten-plate where the maximum stress observed is 18 400 lb. per sq. in., or about 30% of the column stress. Even at the transverse diaphragm the stresses in the lattice-bars were very small, showing the same outward bulging as at other points.

The columns of the next two types, having transverse bars, show considerable compressive stresses in all lattice-bars and tensile stresses in all transverse bars. The tensile stresses in the batten-plates (in a transverse direction) are relatively large. These results show the general tendency of transverse bars to maintain the main ribs in line better than latticing without transverse bars and thus to prevent some of the bending stresses which occur in the latter type near the batten-plates. Neither the longitudinal diaphragm nor the transverse diaphragm appear to have had much effect on this action.

The strain-gauge measurements on the lattice system of Column US_2MS_1 and Column LC_0LC_1 were not sufficient to afford a detailed study of the stresses. From the measurements made on Column US_2MS_1, it is apparent that the lattice-bars suffered considerable flexure in their own plane, also that the average unit stress in these bars was small. The manner of failure of Column LC_0LC_1 (carbon) and Column LC_0LC_1 (Mayari) suggests that the lattice system was insufficient to develop integral action between the several ribs, and that there was considerable secondary bending stress caused at the ends of the lattice-bars by the rigidity of the riveted connections. The ratio of ultimate strength to yield point, as shown in Column (4) of Table 35, is a maximum for Type 1, and nearly uniform for the various columns of Types 2 and 3. The last type was relatively weak, due apparently to the weakness of the lattice system, no longitudinal web being present to assist in carrying the shear.

Additional information concerning the lateral bulging of column ribs under the pantograph action of a double lattice system and the constrictions produced by tie-plates at the ends is afforded by measurements on Columns $TC4_2$, $TC5_2$, and $TC6_2$, which were tested in 1912 for the New Quebec Bridge.[33] Columns $TC4_2$ and $TC5_2$ consisted of two built-up channel sections with transverse diaphragms at the pin-plates and double lattice without transverse bars. Column $TC5_2$ had some intermediate diaphragms also. Each rib in these columns consisted of two 4 by 4-in. by ⅝-in. angles turned outward and a 22-in. by ¾-in. web-plate. The lattice for Column $TC4_2$ consisted of 2½-in. by $\frac{7}{16}$-in. bars; for Column $TC5_2$ it was made of 4-in. by $\frac{7}{16}$-in. bars.

Column $TC6_2$ had two I-shaped ribs each made of four 4 by 4-in. by ⅝-in. angles united by a 24-in. by ⅝-in. web-plate. The ribs were connected along their axes by a longitudinal diaphragm, 16 in. by ⅝ in. in section, which was attached to the ribs by four 4 by 3-in. by ⅝-in. angles. A double-lattice system made of 3-in. by ¼-in. bars was used to tie each pair of flanges. Transverse diaphragms were also supplied at intervals along the column.

By means of the strain measurements taken on transverse gauge lines separated about 1 ft. along the axis of the columns, it was found that spreading at the edge of the tie-plate for Columns $TC4_2$ and $TC5_2$ amounted to about 0.003 in., and at 1½ ft. from the plates, it was about 0.01 in. In Column $TC6_2$, equipped with the longitudinal diaphragm, the strains were about one-fourth of these values. The measurements showed that the ribs were bent concave outward at the tie-plates and convex outward for the length of the first panel of lattice. Calculations of stress showed that the bending stress at the tie-plate augmented the direct compression in the ribs of Columns $TC4$ and $TC5$ by about 20 per cent.

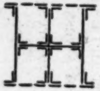
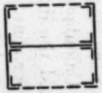
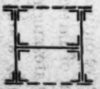
55.—The Effect of Diaphragms on Column Strength.—

Longitudinal Diaphragms.—A good example of the value of a longitudinal diaphragm is afforded by a comparison of Columns $TC4$, $TC5$, and $TC6$ of the

---

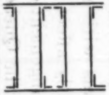
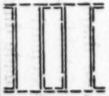
[33] See The Quebec Bridge Report, Vol. 1, p. 207.

TABLE 35.—UNIT STRESSES IN THE LACING SYSTEMS OF LARGE BRIDGE COLUMNS.
(*Technologic Paper No. 101*, U. S. Bureau of Standards. Unit stresses in Columns (10) to (14) were calculated from $S = 29\,500\,000 \times$ unit strain; t = tension; c = compression.)

Column No.	Shape of cross-section.	Area of cross-section, in square inches.	Ratio, $\frac{\text{Ultimate strength}}{\text{Yield point}}$	BRACING OF COLUMN.				UNIT LOAD, IN 1 000 POUNDS PER SQUARE INCH ON					
				Lattice.	Transverse diaphragm.	Longitudinal diaphragm.	Battens.	Column, $\frac{P}{A}$.	Lattice.		Transverse Bars.		Battens, average.
									Maximum.	Mean.*	Maximum.	Average.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
U_2U_8 U_8U_9		88.8	1.11 1.11	2½-in. channels, ¼-in. web.	Three ¼-in. plates, three 5/16-in. plates.	5/16-in. plates.	¼-in. plates.	58.0 32.0 58.0 30.0	18.4c 6.7c 9.6c 3.7c	....	....	....	....
HC *MY* 31*HC* 32*S* 34*CS*		74.6 74.9 76.9 76.9 76.9	0.94 1.03 0.98 0.92 0.88	Transverse 4 × 3 × 5/16-in. angles, diagonal 4 × ¼-in. bars.	Three ¼-in. plates.	7/16-in. plates.	¼-in. plates.	.... 45.0 30.0 .... 30.0	..... 11.9c 9.0c 6.5c	.... 8.3c 5.0c 4.2c	.... 8.8t 5.4t 6.3t	.... 7.6t 4.5t 4.8t	.... 18.4t 12.4t 11.1t
*HC*1 *MY*1 31*HC*1 32*S*1 34*CS*1		75.4 75.3 77.2 77.2 77.2	1.03 1.04 1.09 0.92 0.96	Transverse, 4 × 3 × 5/16-in. angles diagonal 3 × ¼-in. bars.	Three ¼-in. plates.	7/16-in. plates.	¼-in. plates.	.... 40.0 30.0 .... 40.0 30.0	.... 13.2c 7.2c	.... 9.9c 6.2c	.... 7.4t 6.6t 10.3t 10.0t	.... 4.7t 3.8t 8.4t 7.4t	.... 17.0t 11.4t 14.8t 10.2t

* Algebraic mean.

TABLE 35.—(*Continued.*)

Column No.	Shape of cross-section.	Area of cross-section, in square inches.	Ratio, $\frac{\text{Ultimate strength}}{\text{Yield point}}$.	BRACING OF COLUMN.		UNIT LOAD, IN 1 000 POUNDS PER SQUARE INCH ON							
				Lattice.	Transverse diaphragm.	Longitudinal diaphragm.	Battens.	Column, $\frac{P}{A}$	Lattice.		Transverse Bars.		Battens, average.
									Maximum.	Mean.	Maximum.	Average.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
$US_2MS_1(C)$		56.0	*	Transverse tie-plates ¼-in., diagonal 3 × ¼-in. bars.	Four ¼-in. plates.	....	5/16-in. plates.	25.0	10.7c	....	....	....	....
$LC_0LC_1(C)$		118.2	0.78	Tie plates ¼-in., diagonal 3 × 1¾ × ¼-in. angles.	Four ¼-in. plates.	....	5/16-in. plates.	30.0	†2.6c	....	....	....	....

* Failed by lateral bending of forked ends.
† Only one bar measured.

Quebec Bridge tests. The general make-up of these columns has already been described in discussing stresses in lattice. The lengths of these columns were the same; all were equipped with double lattice and transverse bars; but only Column *TC*6 had a longitudinal diaphragm. The principal results of these tests are summarized in Table 36. Each result represents two tests.

TABLE 36.—Comparison of Columns, Quebec Bridge.

Column No.	Ratio, unsupported width of web-plate to thickness (estimated).	Relative ratio, weight to cross-sectional area.	Ratio, ultimate strength of column to yield point of metal.	Relative strengths in proportion to weight.
*TC*4	23	1.00	0.566	1.00
*TC*5	23	1.05	0.559	0.94
*TC*6	21	1.09	0.669	1.08

Mention has already been made of the effect which the longitudinal diaphragm exerted to lessen the lateral bulging of the flanges at the end batten-plates in Column *TC*6.

Attention is also called to the comparisons in Table 37. Although there are many variables other than the effect of the longitudinal diaphram, which affect these comparisons, yet it seems probable that the influence of the diaphragm was the most important factor in raising the efficiency of the column in which it was used.

In test of Column 31*HC*1 (*Technologic Paper No. 101,* U. S. Bureau of Standards), measurements were made of the axial and transverse strains in the longitudinal diaphragm at the middle of a panel where the transverse tension in the diaphragm would be expected to be greatest. At $P/A = 30\,000$ lb. per sq. in., the unit axial compression was 0.00111, and the unit lateral extension was 0.000375. If Poisson's ratio was 0.30 and $E = 29\,000\,000$ lb. per sq. in. for this column, then the axial compression was 31 600 lb. per sq. in. and the transverse tension, 2 340 lb. per sq. in. These results show that the diaphragm was participating in carrying the load and also sustaining a small amount of transverse tension. The ribs were spaced 16½ in. on centers, the diaphragm was $\frac{7}{16}$ in. thick, and a double lattice of 3 by ¼-in. bars, with 4 by 3-in. by $\frac{5}{16}$-in. angle transverse ties, was used. Assuming the tensile stress uniform over a length of diaphragm equal to the rib-spacing, the total transverse pull in that portion was $16.5 \times \frac{7}{16} \times 2\,340 = 16\,900$ lb., or about 0.73% of the column load

Stresses at Transverse Diaphragms.—The ratio of the spacing of transverse diaphragms to the unsupported widths of the adjoining web-plates, and the position of these diaphragms with respect to the lattice intersections, are factors which probably greatly affect the stresses near the diaphragms. Data containing a systematic arrangement of these variables are wanting, but strain measurements made during the Quebec Bridge tests are of importance in this connection.

Columns *TC*4 and *TC*5 were equipped with transverse diaphragms which intersected the ribs at panel points of the lattice. Readings of strain in the outer fibers of the rib flanges showed that the compressive stresses on these fibers for gauges spanning the diaphragm were from 7 to 14% higher than the stresses in adjoining gauges on the same fibers. These data show that the diaphragm offered restraint against the outward thrust of the lattice-bars. Consequently, the rib flanges were thus bowed concave outward at the diaphragm in much the same way as at the end battens. Measurements on Column *TC6* which had I-shaped ribs and a longitudinal diaphragm, failed to show any marked discrepancy between strains across the transverse diaphragms and elsewhere.

TABLE 37.—SHOWING COMPARATIVE STRENGTHS OF LARGE BUILT-UP COLUMNS WITH AND WITHOUT LONGITUDINAL DIAPHRAGMS.

Column No. (1)	Report. (2)	$\frac{l}{r}$ (3)	Ratio, $\frac{\text{Unsupported width of web-plate}}{\text{Thickness (estimated)}}$ (4)	Ratio, $\frac{\text{Ultimate strength}}{\text{Yield point of material, } A}$ (5)	Mean ratio, *A*. (6)	Type of failure. (7)
			COLUMNS WITH DIAPHRAGMS.			
U_2U_8	*Technologic Paper No. 101*, U. S. Bureau of Standards.	25	9	1.11	....	Flanges buckled
U_8U_9		37	9	1.11	....	" "
HC		44	7	0.94	....	Buckled at center
MY		44	7	1.03	....	Bending
31*HC*		44	7	0.98	....	"
32*S*		44	7	0.92	....	Center web buckled
34*CS*		44	7	0.88	....	Center web and lacing buckled
*HC*1		36	10	1.03	....	Bending
*MY*1		38	10	1.04	....	"
31*HC*1		38	10	1.09	....	Diaphragm buckled
32*S*1		38	10	0.92	....	Center web buckled
34*CS*1		38	10	0.96	....	Buckling and bending
*TX*13*A*1	Quebec Bridge.	38	14	0.97	....	Flanges at splice
*TX*13*A*2		38	14	1.00+	....	No failure
*TX*13*B*1		38	14	1.00+	....	" "
*TX*13*B*2		38	14	1.00+	....	" "
*TX*17-1		78	34	0.78	....	Ribs buckled at splice
*TX*17-2		78	34	0.79	....	" " " "
*TX*18-1		58	34	0.83	....	Bending
*TX*18-2		58	34	0.86	....	"
					0.96	
			COLUMNS WITHOUT LONGITUDINAL DIAPHRAGMS.			
$LC_0LC_1(C)$	*Technologic Paper No. 101*, U. S. Bureau of Standards.	28	13	0.78	....	Plates and angles buckled
$LC_0LC_1(M)$		27	24	0.93	....	" " " "
*TX*16*C*1	Quebec Bridge.	52	17	0.90	....	Ribs buckled at splice
*TX*16*C*2		52	17	0.81	....	" " " "
*TX*16*N*1		52	11	0.81	....	Ribs buckled
*TX*16*N*2		52	11	0.88	....	" "
No. 5	Howard.	47	19	0.86	....	Web-plate buckled
					0.85	

In Column U_2U_3 (*Technologic Paper No. 101,* U. S. Bureau of Standards), the diaphragms were placed at the intersection of the lattice (mid-points of panels) at intervals of two panel lengths. In Column U_8U_9, the end diaphragms were similarly placed, but the center diaphragm was attached at panel points of the lattice. Measurements of strain along the edges of the center diaphragm in the plane of the lattice gave the results shown in Table 38. It will be noted that the stresses are somewhat greater in the $\frac{5}{16}$-in. diaphragm of Column U_8U_9 than in the $\frac{1}{4}$-in. diaphragm of Column U_2U_3. This discrepancy is probably due to the difference in the positions of the diaphragms with respect to the lattices.

TABLE 38.—Strains in Diaphragms.

Column No.	Unit load, P/A, in pounds per square inch.	Average unit strain on edge of diaphragm.	Calculated unit stress in diaphragm, in pounds per square inch.
U_2U_3	10 000	0.00027	8 100
	32 000	0.00103	30 500
	44 000	0.00155	45 500
U_8U_9	10 000	0.00039	11 400
	30 000	0.00125	37 000
	45 000	0.00186	55 000

Summary of Results

The results of tests and the theoretical studies contained in this report may be summarized briefly, as follows:

(1) When correctly interpreted, the average tensile yield point of the material is a satisfactory measure of the ultimate strength of columns, except for columns having high values of the slenderness ratio, in which buckling takes place at stresses below the yield point.

(2) The shearing rigidity of columns varies greatly as between columns with solid webs, columns with lacing, and columns with batten-plate connections.

(3) The strength of single lacing-bars in resisting column shear can be determined approximately by tests on single bars, in the manner set forth in the report.

(4) Columns the component parts of which are connected by batten-plates are relatively weak in shear, and under loads involving heavy shearing stress, will be subjected to relatively high stress in the extreme fibers of the individual segments. Under loads producing small shearing stresses, the tests are not conclusive as to the economy of batten-plate columns.

(5) Under loads the eccentricities of which are known, the results of tests check very closely with theoretical results.

(6) Eccentric load tests on **H**-columns showed a very considerable excess of ultimate strength over that determined on the basis of yield point in the case where the eccentricity was in a direction perpendicular to the web, so that the most stressed fibers were on the edges parallel to the web. This excess strength was, in general, greater the smaller the slenderness ratio, $\frac{l}{r}$.

(7) On a theoretical basis, the flexibility of a column in the plane of the lacing due to shearing distortion is not of great influence on the ultimate

strength of columns under eccentric loads. Such reduction of strength as may be caused by this increased flexibility can be overcome by an increase in the corresponding radius of gyration.

(8) The results shown in the Progress Report of 1926 to the effect that the tensile and compressive yield points are very nearly equal, are confirmed by the tests herein described.

(9) The yield point of the material varies considerably according to the location of the specimens in the section. For **H**-columns, the specimens cut from the intermediate portions of the flanges appear to represent the section best. For channels, the web specimens were most representative. For thin plates, not much difference was found between specimens cut from the edges and those cut from the center.

(10) The value of the modulus of elasticity of the column as a whole is nearly constant, ranging only about 5% more or less than 29 500 000 lb. per sq. in.

(11) The theoretical analysis of a column forming a part of a riveted truss indicates that the strength of the column as regards bending in the plane of the truss is much greater than that of a pivoted end column, and approaches rather closely the strength of a pivoted end column of one-half the length under eccentric load, with eccentricity corresponding to the secondary bending moments assumed in the analysis.

(12) In the case of columns forming compression members of riveted structures, the effect of eccentricity due to crookedness and the effect of weight as regards bending in the plane of the truss are relatively small, the effect of crookedness being less than one-half that due to the same amount of crookedness in a pivoted end column of one-half the length.

(13) The crookedness of the columns tested was, in most cases, less than 1/1500 of the length. All columns were fabricated under standard American Railway Engineering Association specifications.

(14) The obliquity of the ends of columns designed to transmit largely by end-bearing is a matter of importance. As measured by the slope of one end with respect to the other, the average obliquity of the **H**-columns tested was found to be about 0.003.

(15) A study of the existing data relative to the failure of individual column components shows that in laced channel columns, no failures occurred where the ratio, $\frac{p_1}{r_c}$, was less than 25. The tests studied were, however, too limited in number to be at all conclusive.

(16) The tests made by the U. S. Bureau of Standards for the Delaware River Bridge Commission indicate that a safe ratio of plate width to thickness for structural steel is about 40, provided the width is measured between rivet lines and the plates are well restrained at the edges.

(17) Under perfect conditions of support at one edge, outstanding structural steel plates or flanges may be expected to carry the full proportional limit stress of about 30 000 lb. per sq. in. without buckling, provided the ratio of outstanding width to thickness is not greater than 20.

(18) The stresses in lacing are likely to be considerably greater than those due to calculations from shear; in centrally loaded columns (shear = 0) generally they are greater by an amount from 0.015 to 0.02 P, and in obliquely loaded columns, by 0.015 to 0.03 P. For loads approaching the maximum, the stresses in the lacing-bars approach more nearly the calculated values.

(19) The action of double lacing without transverse bars forces the ribs of a column apart, and introduces considerable bending stresses near the stay-plates. With transverse bars and double lacing, this action can be reduced, but the lacing will then receive considerable compressive stress, and this arrangement involves details which are undesirable in some respects.

(20) Tests on large bridge members indicate difficulty in securing integral action of the several ribs of a large member by means of lattice-bars only. Longitudinal diaphragms or cover-plates are more effective.

SUGGESTIONS RELATIVE TO FUTURE WORK

From the studies made thus far, it seems evident that further investigation of columns may well be considered in two parts: (1) The strength and effectiveness of lacing, batten-plates, and diaphragms; effect of form of section, outstanding flanges, etc.; unsupported length of component parts; and other details affecting integrity of action; and (2) the conditions of the column as a part of a structure, its end conditions, crookedness, and the effect of other members thereon.

The first group of studies can best be made with end conditions definitely known, as by the use of roller-bearing blocks; the second group can only be made by studies and observations of actual columns and trusses. If details are designed to secure with certainty integrity of action, and then if the conditions of the column in the truss are known, it would seem that the application of theoretical principles is the rational process of design and is quite as well warranted as in the design of beams. The attempt to study the entire problem by the customary flat-end test is a slow process, and does not yield results that are readily analyzed and applied.

Further needed information can be secured regarding the first group of problems by a moderate expenditure for test specimens and laboratory work; the second part of the problem is more difficult, and will probably require a considerable amount of field work.

Respectfully submitted,

F. E. TURNEAURE, *Chairman*,
C. G. E. LARSSON,
B. R. LEFFLER,
G. L. TAYLOR,
S. H. WIDDICOMBE,
Special Committee on Steel Column Research.

May 7, 1928.

DISCUSSION

Henry S. Prichard,[34] **M. Am. Soc. C. E.** (by letter).—A consideration of the progress reports, made in January, 1926,[35] and January, 1929, by the Special Committee on Steel Column Research, excites admiration for extensive, painstaking, able analyses, and gives ground for hope that much of practical value will result from the research after the contemplated further investigations have been completed. The final report will be awaited with much interest.

The theory of flexure is based on the proportionality of stress to strain and on the modulus of elasticity being constant for the material to which the theory is applied. When it is applied to a body as a whole, it is assumed that the details are properly proportioned. In general, the theory is applied as if friction between the bodies or parts to which it is applied and other bodies or parts is negligible. It is gratifying to find so much verification of theory in the physical investigations within these qualifications. It is also gratifying that the modulus of elasticity for steel proved to be so nearly uniform for all cases observed, and that the mathematical analyses based on proportionality of stress to strain agree with experimental determinations, as far as the general behavior of columns is concerned, over such a considerable range of stress.

Engineers and architects, for many years, have based their practice with regard to beams on the ordinary theory of flexure, and during the same period have based their practice as regards columns on arbitrary formulas, some being little more than rule-of-thumb methods modified more or less by experiments and experience, some being the result of mistakes in theory, and some being based on pseudo theories or mere notions. Yet the ordinary theory of flexure applies more closely to columns than to beams and, intelligently applied, finds better confirmation in the case of columns than in that of beams. This is due in part to the facts that the ordinary theory of flexure neglects the influence of shear on deformation and shear is more of a factor in beams than in columns. The permanent sets under transverse loads indicated in Column (6) of Table 5 may have been due to the transverse load at the center being almost concentrated on a line. The ordinary theory of flexure, which neglects the influence of shear on deformation, gives no indication, in the cases tabulated, of stresses intense enough to produce permanent sets. When the influence of shear is considered, however, it indicates that very high stresses, much higher than those indicated by the ordinary theory, may occur if the transverse load is too highly concentrated.[36]

Under ideal conditions a centrally loaded column with frictionless hinged ends would be subjected to no shear; but in order that the various fibers constituting the column shall act together, it must have potential resistance to shear. This potential resistance has been neglected in Euler's formula, in consequence of which that formula gives a result which is, theoretically, slightly too great, but only very slightly so for a column with a reasonable cross-section.[37]

---

[34] Pittsburgh, Pa.

[35] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1485.

[36] *Loc. cit.*, p. 1257, Equation (22) ; see, also, Vol. LXXV (1912), pp. 932–953, for effect of shear on deformation of beams.

Section 35, advocating "Factor of Safety Method in Column Design", is highly commendable and should receive special consideration. If rational methods of computing the relation between loads and stresses and shears are used, then, actually to obtain, in a column, the factor of safety desired, it must be applied to the critical load and not to the critical stress or shear, the critical load being the load which would produce the critical stress or shear; that is, the stress or shear at or near failure.

Quoting from Section 35:

"* * * the logical process in column design is to select a formula which will as nearly as practicable give values of ultimate strength, or some point which represents the practical ultimate value of the column; then to divide the resulting value of average stress, P/A, by the factor of safety corresponding to that used in other members* * *".

This leaves an open question as to just what point in loading should be considered the critical point to which the factor of safety should be applied. In its First Progress Report the Committee stated[38] that:

"The Society's Special Committee on Steel Columns and Struts made use of what was named by it the 'Useful Limit Point' (U. L. P.)[39] as being of more significance in some respects than the ultimate strength of the column or the yield point of the material. Although there is good ground for adopting this point in the stress-deformation diagram, as the best measure of column strength, it is not practicable to adhere to this method in a study of all available data. Information does not exist to determine the U. L. P. in a great many of the tests that have been made; but, nevertheless, it is believed that the results of such tests are valuable and cannot be neglected."

The Committee further stated:[40]

"It is possible that some such point as the U. L. P. used by the Society's Special Committee on Steel Columns and Struts would be a more accurate reference point than the yield point, but as no other tests give this information,[41] it seemed best to use the yield point instead, and it is believed that the diagrams represent fairly the relations indicated.

"The difficulty of securing correct information regarding the elastic properties of the material actually used is well known. The values of yield point as determined by the usual 'drop of the beam' method depend largely on the details of testing; the mill tests vary largely from laboratory tests; and the material varies greatly in thin and thick sections, and at different points in the same section."

The writer's opinion of the critical value of a structural steel member in direct compression or tension, in the formation of which he gave great weight to the report of the Special Committee on Steel Columns and Struts, has been published.[42]

---

[37] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1423; see, also, *Engineering News*, February 25, 1909, p. 206.

[38] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1487.

[39] *Loc. cit.*, Vol. LXXXIII (1919–20), p. 1618.

[40] *Loc. cit.*, Vol. 89 (1926), p. 1522.

[41] In numerous tests contained in various Watertown Arsenal reports the strains near the U. L. P. were recorded at short intervals of stress. In each of such cases, the U. L. P. can be closely approximated.

[42] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), pp. 1236 and 1239.

In ordinary cases the only criteria which an engineer or architect has of the strength of the steel in the structure he has designed, are the specimen tests, furnished by the mill, of the tensile yield point, as determined by "drop of beam", and the ultimate tensile strength. The "yield point" as furnished by the mill is often much higher than that found in laboratories, such as that of the U. S. Bureau of Standards, and is usually much higher for specimens than for full-sized structural members.

The U. L. P., the yield point, and the ultimate strength of different rolled sections, as found in properly conducted tests in reliable laboratories, differ greatly for steel nominally of the same quality, even from the same heat. For instance, in the careful tests by Marshall, the "elastic limit", as he terms it, taken by "drop of beam", of a specimen from a 4-in. by ½-in. flat, was 50% greater than that of a specimen from a 3 in. by 1¾-in. flat, although they were from the same blow of Bessemer steel.[43] The extremes in the characteristic U. L. P.'s of the columns made of 60 000 lb. (desired) steel, tested for the Society's Special Committee on Steel Columns and Struts, were 19 000 and 35 000 lb. per sq. in., respectively.[44]

The great differences in utilizable strength due to the thermal and mechanical conditions encountered in the normal processes of manufacture, as distinguished from differences in chemical composition, overshadow the differences due to length, radius of gyration, and form of cross-section, in cases where the conditions are such that the capacity of the column can be gauged more by its resistance to stress than by its degree of stiffness. In long columns, the intensity of elastic stress which steel will resist is not so much of a factor.[45]

The members of framed structures, even pin-connected ones, have secondary stresses, in addition to direct stresses. The direct stresses are generally computed as if the structure were perfectly articulated and the differences between the combined stresses and the direct stresses are considered secondary stresses. The determination of the secondary stresses is a complicated matter. It requires for each member a consideration of all the members, including their section and make-up, and, from a practical point of view, is only approximate, on account of the complication due to connections at the ends of the members. Apart from these considerations, as the determination of secondary stresses is based on the proportionality of stress to strain, it can only be correct within the limits to proportionality. If carried beyond these limits it is liable to be very faulty.

In view of the uncertainties as to the intensity of stress to which steel structures are liable in many of their details and of uncertainties as to the elastic resistance to stress possessed by the numerous pieces of steel used, it is fortunate that steel possesses capacity to recover elasticity—albeit with permanent set—after it has been subjected to plastic deformation in service. This phase of the subject should and probably will form part of the research of the Committee before it has concluded its labors.

---

[43] *Transactions*, Am. Soc. C. E., Vol. XVII (1887), Table 1, p. 68(*a*).

[44] *Loc. cit.*, Vol. 89 (1926), p. 1268.

[45] *Loc. cit.*, p. 1292, Fig. 6.

The many phases involved in column research should be kept in mind in judging the reports of the Committee; otherwise, too much will be expected. Many uncertainties are inevitable under the conditions of manufacture, testing, and use which now exist.

E. H. SALMON,[46] ESQ. (by letter).—The Committee is to be congratulated on its second Progress Report, which contains much information worthy of the closest study. The writer is convinced that important progress will develop from a detailed investigation of the behavior of columns, particularly those forming part of a truss. In the ordinary structural column, length-strength considerations do not have the preponderating importance which has been attached to them in the past.

The Committee refers to some estimates by the writer of the imperfections, initial curvature, etc., likely to occur in practical columns, and the statement is made that the measured imperfections in specimens used for the report were considerably less. It is freely admitted that there is nothing sacrosanct about the writer's estimates, which were based on the data available in 1915. Nevertheless, what the Committee's observations really prove is that its specimens were very well straightened. Although the columns were required to conform to the General Specification for Railway Bridges of the American Railway Engineering Association, it would, perhaps, be rather natural if a little extra care were taken with members intended for such a special purpose. However this may be, it is of interest to compare the Committee's suggested formula, $e' = \frac{l}{1\,500}$, with earlier observations. The writer has reached the conclusion that there is no evidence to show that the initial deflection is a function of $\frac{l}{r}$ rather than of l, with which the Committee seems to agree. The larger diagram in Fig. 40 is the original from which the rule, $e' = \frac{l}{750}$, was deduced; a line representing the Committee's suggestion, $e' = \frac{l}{1\,500}$, has been added. It is found by counting that of the 185 plotted points, 7 lie outside the line, $e' = \frac{l}{750}$, and 78 (42%) lie outside the line, $\frac{l}{1\,500}$. It is submitted that this number is much too large to be fortuitous, and that, in practice, it would not be feasible to rely on securing a crookedness as slight as $\frac{l}{1\,500}$. Examining Fig. 37 in the same way: Of the 39 plotted points, 5 of them, or 13%, are outside the line, $e' = \frac{l}{1\,500}$. It is to be observed that most of these specimens were about 100 in. long, and that in the fewer long specimens, the initial deflection was small, whereas in Fig. 40 the percentage in which $\frac{l}{1\,500}$ is exceeded, is greater for the greater lengths.

---

[46] Clark & Standfield, London, England.

Taking Fig. 37 in conjunction with Fig. 40, it appears to the writer that, with the data available, the estimate, $e' = \frac{l}{750}$, is not unreasonable. Nevertheless, what he would like to see would be the measurement (as suggested in the Committee's previous report[47]) of a large number of actual members (not test specimens) as fabricated ready for erection, made at different yards, and classified as to size and shape; that is, small single bars differentiated from large built-up members. The cost should not be excessive, and if from these data, the Committee could fix a standard which might pass inspection in the shops, he conceives that one debated point in column design would have been settled. This is a matter in which the large steel fabricating companies might co-operate usefully to a common advantage.

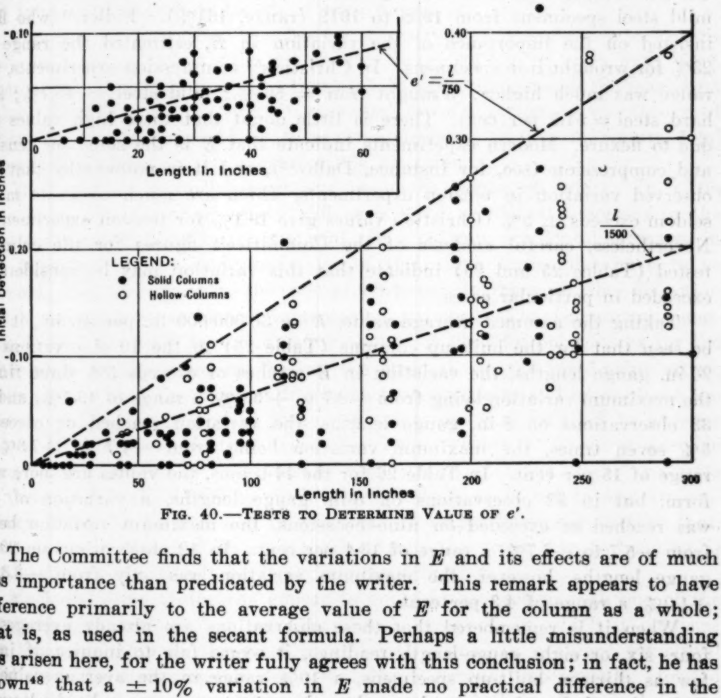


FIG. 40.—TESTS TO DETERMINE VALUE OF e'.

The Committee finds that the variations in E and its effects are of much less importance than predicated by the writer. This remark appears to have reference primarily to the average value of E for the column as a whole; that is, as used in the secant formula. Perhaps a little misunderstanding has arisen here, for the writer fully agrees with this conclusion; in fact, he has shown[48] that a ± 10% variation in E made no practical difference in the value of EI, provided the average value was used for E. It is of importance, however, to recognize the effect of a difference in the value of E for the two flanges of a built-up member, for example, in shifting the center of resistance away from the geometrical center of the cross-section. It is easy to prove that a ± 10% variation in E shifts the center of resistance, $\frac{h}{20}$, from the geometrical center, where h is the distance between the centroids of the two flanges.

---

[47] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1537.

[48] "Columns," by E. H. Salmon, Oxford Technical Publications, p. 34.

With the data available prior to 1915, the writer estimated the probable variation in E as ± 10 per cent. The Committee finds for the built-up columns a range of 10% (— 6.6 to + 3.8%). It is quite probable that the previous estimates for the values of E in compression are inaccurate. The determination of E in compression experiments is not easy. Strain-gauge measurements are really not very suitable, and the effect of flexure is probably always present. The writer based his estimate chiefly on Bauschinger's[49] experiments on wrought-iron specimens, which were very carefully made (range, 22½%), and on fifteen experiments made at Watertown Arsenal[50] on mild steel specimens from 1908 to 1912 (range, 16½%). Fidler,[51] who first insisted on the importance of the variation in E, estimated the range as 23% for wrought-iron specimens. In Christie's[52] compression experiments, the range was much higher: Wrought iron = 56½%; mild steel = 45.7%; and hard steel = 51.2 per cent. There is little doubt that these high values are due to flexure. Modern experiments indicate that E is the same for tension and compression (see, for instance, Dalby[53]), and it is noteworthy that the observed variation in tension experiments, which are much easier to make, seldom exceeds ± 5% (Christie's values give 19.1% for tension experiments). Nevertheless, careful analysis of the Committee's figures for the columns tested (Tables 25 and 26) indicate that this variation may be considerably exceeded in particular cases.

Taking the assumed average value, E = 30 000 000 lb. per sq. in., it will be seen that for the built-up columns (Table 25) in the 10 observations on 20-in. gauge lengths, the variation in E reaches or exceeds 5% three times, the maximum variation being from —8.7 to + 5.0%, a range of 13.7%; and in 33 observations on 8-in. gauge lengths, the variation reached or exceeded 5% seven times, the maximum variation being from —7.7 to +7.3%, a range of 15 per cent. In Table 26 for the H-beams, the values are more uniform, but in 52 observations on 8-in. gauge lengths, a variation of 4% was reached or exceeded on nine occasions, the maximum variation being from —5.7 to +7.7%, a range of 13.4 per cent. In 12 observations on 20-in. gauge lengths, however, the maximum variation was only from —3.3 to +1.0%, a range of 4.3 per cent.

When it is remembered that these observations are already averages of four, six, or eight gauge-length readings, it seems fair to inquire, if in as few as thirteen built-up specimens, a 10% range in the average value of E for the whole column is observed, and a 15% range on individual gauge lengths, whether it is very extravagant to suppose that a possible 20% difference in E might occur between the two flanges of a built-up column in practice. Presuming that the ends of such a member were fixed in direction, and the writer's suggestion (that one-half the shift of the center of resistance be regarded as initial curvature, and one-half as eccentricity) is adopted, the eccentricity would not affect the strength, and the proposal would be equivalent to taking the 10% range found by the Committee entirely as initial curvature. Even so, if such a member were short, the effect of the difference in modulus would be the larger part of the initial curvature. In this connection it may be recalled that Fidler took the variation in the modulus as the sole imperfection.

---

[49] "Zerknickungs-versuche," *Mitt.* a. d. Mech. Tech. Lab. d. k. Tech. Hochschule in München, Heft 15, Mitt. XVIII, p. 11.

[50] Reports of Tests of Metals, Washington, D. C., 1909–13.

[51] "The Practical Strength of Columns," *Minutes of Proceedings*, Inst. C. E., Vol. LXXXVI, p. 261, 1886; and "A Practical Treatise on Bridge Construction," Lond., First Edition, 1887.

[52] "Experiments on the Strength of Wrought-Iron Struts," *Transactions*, Am. Soc. C. E., Vol. XIII (April, 1884), p. 85; (August, 1884), p. 253.

[53] "The Mechanical Properties of Steel," *Minutes of Proceedings*, Inst. C. E., Vol. CCXXI, p. 21, 1925–26.

It would be of much interest if the Committee would give the actual difference in E between the two flanges of these thirteen columns, averaging the value over the whole length of each flange.

The question immediately arises as to whether it is necessary to assume that the member suffers from all possible imperfections at once, and, if so, whether it is necessary to assume that each imperfection reaches the maximum value. Professor R. H. Smith[54] suggested an application of the theory of probabilities, and proposed to take seven-elevenths of the maximum total eccentricity, acting in the worst possible direction.

The question is really dependent upon the factor of safety. In a tie or a beam, one factor of safety will cover both accidental increases in the load and imperfections in the material. In a column, accidental increases in the imperfections, or under-estimates of their magnitude, also have to be covered, and, in addition, the non-proportional increase in the deflection which accompanies accidental increases in the load. The Committee's suggestion that the same factor of safety should be used for columns as for the other parts of the structure, basing the strength on the yield-point stress, really implies that the true conditions as to eccentricity and crookedness are known; and unless a special factor of safety is to be applied to average values of these imperfections (as some have proposed), to cover possible exceptional material or exceptionally bad straightening, a liberal estimate of their magnitude should be made—such, indeed, as is not in any circumstances likely to be exceeded in practice. It might be urged further that, since in the ordinary short structural column the stress due to bending is small, an over-estimate is not a serious consideration.

Much the same arguments appear to apply in connection with one of the most significant results of the Committee's research, in which it confirms the large variation in the yield point which occurs in ordinary structural mild steel. In discussing the later Watertown Arsenal experiments, Howard repeatedly calls attention to similar variations, and their effect in bringing about local failure in built-up members.

While the Committee may fairly be said to have established that, for practical purposes the tensile yield point may be taken as the failure stress, either a low minimum value should be taken as the yield stress, or if the average value is taken, a reduction of 15 to 20% should be allowed to cover the possible variations. In other words, the Committee's results lend support to the rather usual practice of adopting a lower working stress in compression than in tension.

---

[54] "The Strength of Struts," *Proceedings*, Edinburgh and Leith Eng. Soc., 1878, Vol. IV, p. 42.

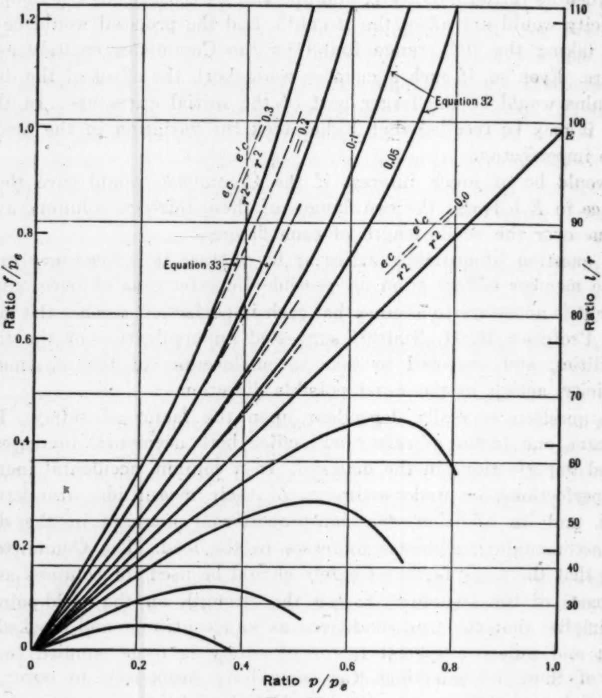


FIG. 41.

Assuming then that a suitable value had been taken for *e*, and that the column has position-fixed ends, the Committee's use of the secant formula to calculate the stress at yield in the column—as indeed is proved by the experiments—is perfectly sound. The method is probably sufficiently accurate in the case of a direction-fixed column provided that a suitable value is taken for the "free length". In seeking an approximation to replace the secant, the writer believes that it is justifiable to take into consideration the fact that, due to the relative shortness of ordinary structural columns, only a small part of the secant curve has any relevance, and that part is very flat. A simple formula can then be used to determine the stress in the column. Fig. 41 has been plotted to illustrate this point. The abscissas represent the ratios of $\frac{p}{p_e}$, and the ordinates, $\frac{f}{p_e}$. Thus, the abscissas represent to some scale the load per square inch on the column, and the ordinates the maximum stress in the extreme fibers. If the column remained absolutely straight up to Euler's crippling load, p would be equal to f, and the maximum stress would be represented by the straight line, *OE*, at 45 degrees. If the column bends, the stress will increase on the concave side and decrease on the convex. This is shown in the heavy curved lines, which are plotted from the formula:[55]

$$\frac{f}{p_e} = \frac{p}{p_e}\left[1 \pm \frac{c}{r^2}\left\{ e \sec \frac{\pi}{2}\sqrt{\frac{p}{p_e}} + \frac{8\,p_e\,e'}{\pi^2\,p}\left(\sec \frac{\pi}{2}\sqrt{\frac{p}{p_e}} - 1\right)\right\}\right] \ldots (33)$$

The first half of Equation (33) will be recognized as the secant formula for eccentric loading, the second half is the corresponding expression for initial curvature. Four cases are shown corresponding to $\frac{e\,c}{r^2} = \frac{e'\,c}{r^2} = 0.05$, 0.1, 0.2 and 0.3, respectively. It is assumed for this purpose that, of the total deviation from axial loading, one-half is due to eccentricity and one-half to initial curvature. The last case is approximately equivalent to $\frac{e\,c}{r^2} = 0.6$ representing, according to Moncrieff,[56] the worst experimental columns, and taken by him as a suitable value for a practical column. On the same diagram is plotted the formula:[57]

$$\frac{f}{p_e} = \frac{p}{p_e}\left[1 + \left(1 + \frac{3\,p}{2\,p_e}\right)\frac{c}{r^2}\,(e + e')\right] \ldots\ldots\ldots\ldots (34)$$

which is represented by the light broken lines. It will be seen that up to the value, $\frac{p}{p_e} = 0.3$, the agreement is excellent; up to $\frac{p}{p_e} = 0.4$, the agreement is good; and even at $\frac{p}{p_e} = 0.5$, for the greatest value of $\frac{e\,c}{r^2} = \frac{e'\,c}{r^2}$, the error is only 10.3 per cent. The ratio, $\frac{p}{p_e}$, is the factor of safety in Euler's formula, that is, against instability, and for ordinary structural columns this is not likely to be less than 5, or $\frac{p}{p_e} = 0.2$; for the shorter columns, it will be much greater. A series of horizontal lines has been drawn on Fig. 41 representing the value of $\frac{l}{r}$, for which, $f = 30\,000$ lb. per sq. in., the minimum value for the yield point of the steel used in the Committee's experiments, plotted from the formula:

$$\frac{f}{p_e} = \frac{f}{\pi^2\,E}\left(\frac{l}{r}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

---

[55] "Columns," Equation (86), p. 42.

[56] "The Practical Column Under Central or Eccentric Loads," *Transactions*, Am. Soc. C. E., Vol. XLV (1901), p. 334.

[57] "Columns," Equation (93A), p. 44.

It will be seen that for the worst case, $\frac{e\,c}{r^2} = \frac{e'\,c}{r^2} = 0.3$, when $f = 30\,000$ lb. per sq. in.; that is, at the failure point, if $\frac{l}{r} = 100$, the value, $\frac{p}{p_e}$, is only equal to 0.46, and if $\frac{l}{r} = 50$, the value, $\frac{p}{p_e}$, is only equal to 0.15.

These values of $\frac{p}{p_e}$ are sufficiently small to justify the use of Equation (34) practically up to the failure point of such columns; and at working stresses the corresponding values of $\frac{p}{p_e}$ will be less than one-half.

Equation (34) is very much simpler than the secant formula, or any of the algebraic substitutes which have been proposed, ingenious as they are, and in view of the fact that a possible variation in f of 33% must be considered and that, at best, e and e' must be probable assumptions, it seems to the writer that the much simpler equation is all-sufficient for practical work provided always the ratio, $\frac{p}{p_e}$, never exceeds 0.5. If it is preferred to treat the deviation entirely as an eccentricity, e, and arrange so that the stress shall be entirely covered by the approximate formula when $\frac{e\,c}{r^2} = 0.6$ and $\frac{p}{p_e} = 0.5$, the constant, $\frac{3}{2}$, may be changed to suit the circumstances, and Equation (34) modified to,

$$\frac{f}{p_e} = \frac{p}{p_e}\left[1 + \left(1 + \frac{5}{2}\,\frac{p}{p_e}\right)\frac{e\,c}{r^2}\right]$$

but for the majority of practical columns Equation (34) can be used with safety.

For columns direction-fixed at their ends (and most practical columns approximate this condition), the curves representing the increase of stress with increasing values of $\frac{p}{p_e}$ are even flatter (see Fig. 42), and a similar device can be applied with even greater justification.

Fig. 42 is plotted differently; the base line represents the ratio, $\frac{p}{p_e}$, as before except that, in this case, p_e is $\frac{4\,\pi^2\,E}{\left(\frac{l}{r}\right)^2}$; but the ordinates represent, to a particular scale, the ratio, $\frac{f_b}{p_e}$, in which f_b = stress due to bending. The curves show that for values of $\frac{p}{p_e}$ from 0 to 0.5 at least, the bending stress in a fixed-ended column increases slowly relative to the increasing load. Incidently, the writer's diagrams illustrate the fact that for small values of $\frac{p}{p_e}$, that is, under working conditions, the stress due to bending at the ends is double that at the middle, approaching equality when $p = p_e$.

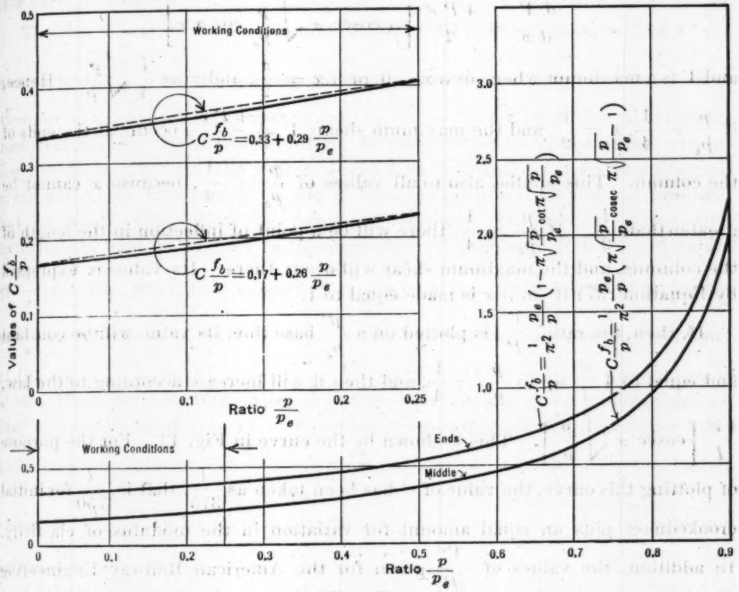


FIG. 42.

If use is made of the suggested algebraic equivalents for the unwieldy trigonometrical expressions, there seems no real reason why the fixed-ended column should not be designed rationally instead of by pivoted-end formulas (which at best are rough approximations), particularly if adjustment were made for possible imperfection in the direction-fixing.

Turning to the Committee's observations on shear for axially loaded columns, the writer considers that this is one of the points in which the "free-length" method of treating fixed-ended columns is apt to be misleading. In a fixed-ended column with parabolic initial curvature, if the origin is taken at its midpoint, the equation[58] for the shear force is:

$$V = \frac{d\,M}{d\,x} = \frac{8\,P\,e'}{\alpha^2\,l^2}\left(\alpha\,k\frac{\alpha\,l}{2}\operatorname{cosec}\frac{\alpha\,l}{2}\sin\alpha\,x\right) \ldots\ldots\ldots\ldots (36)$$

in which, $\alpha = \frac{2\,\pi}{l}\sqrt{\frac{p}{p_e}}$; $p_e = \frac{4\,\pi^2\,E}{\left(\frac{l}{r}\right)^2}$; and for a column with perfectly direction-fixed ends, $k = 1$. This reduces to:

---

[58] "Columns," Equation (446), p. 193.

$$V = \frac{4\,P\,e'}{l}\left[\operatorname{cosec} \pi \sqrt{\frac{p}{p_e}} \sin \alpha\, x\right] \ldots\ldots\ldots\ldots\ldots (37)$$

so that,

$$\frac{d\,V}{d\,x} = \frac{4\,P\,e'}{l}\left[\alpha \operatorname{cosec} \pi \sqrt{\frac{p}{p_e}} \cos \alpha\, x\right]$$

and V is a maximum when $\cos \alpha\, x = 0$, or $\alpha\, x = \frac{\pi}{2}$, and $x = \frac{l}{4}\sqrt{\frac{p_e}{p}}$. Hence, if $\frac{p}{p_e} = \frac{1}{4}$, $x = \frac{l}{2}$, and the maximum shear, $V = \frac{4\,P\,e'}{l}$, occurs at the ends of the column. This applies also to all values of $\frac{p}{p_e} < \frac{1}{4}$, because x cannot be greater than $\frac{l}{2}$. If $\frac{p}{p_e} > \frac{1}{4}$, there will be a point of inflection in the length of the columns, and the maximum shear will occur there. Its value is expressed by Equation (37) if $\sin \alpha\, x$ is made equal to 1.

If, then, the ratio, $\frac{V}{P}$, is plotted on a $\frac{p}{p_e}$ base line, its value will be constant and equal to $4\,\frac{e'}{l}$ up to $\frac{p}{p_e} = \frac{1}{4}$, and then it will increase according to the law, $\frac{4\,e'}{l}\left[\operatorname{cosec} \pi \sqrt{\frac{p}{p_e}}\right]$. This is shown by the curve in Fig. 43. For the purpose of plotting this curve, the value of e' has been taken as $\frac{l}{375}$, that is, $\frac{l}{750}$ for initial crookedness plus an equal amount for variation in the modulus of elasticity. In addition, the values of $\frac{V'}{P}$ given for the American Railway Engineering Association experiments on columns with square ends have been plotted on the same diagram. For this purpose the specimens have been considered direction-fixed at their ends. Although the points lie grouped about the curve, too much importance should not be attached to the fact, because the value of

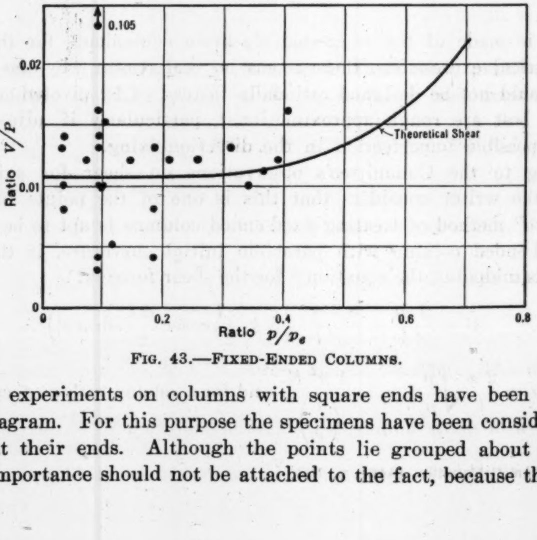


FIG. 43.—FIXED-ENDED COLUMNS.

e' will be different for each, but with the data at his disposal the writer is unable to make a better estimate. It is difficult to believe, however, that in a practical column the ratio of $\frac{V}{P}$ can increase as $\frac{l}{r}$ diminishes, as suggested by the curve in Fig. 39.

The theoretical shear in a column intended to be axially loaded is $V = P\,\frac{d\,y}{d\,x}$, and certainly $\frac{d\,y}{d\,x}$ will be greater in long columns than in short ones. The Committee attributes the larger values of V, as found from the strain-gauge measurements on the shorter columns, to inequalities in the bearings. Granting this to be the case, the writer would regard it rather as an experimental error not likely to occur with the riveted connections adopted in practice; but he would urge that the American Railway Engineering Association experiments lend no support to the suggestion of increased V for small values of $\frac{l}{r}$ (see Fig. 43). This suggestion comes from the two groups of experiments at $\frac{l}{r} = 22$ and $\frac{l}{r} = 38$ (Fig. 39). In the latter group, made at the University of Illinois, the specimens were abnormal, and had pin ends; in the former, made at the University of Wisconsin, the ends were supported on roller bearings. May not the extra degree of end freedom resulting from pin and roller ends have led to an increase in $\frac{d\,y}{d\,x}$, and thus explain, at least in part, the higher values of $\frac{V}{P}$ shown by these experiments? One would like further evidence on this point.

Apart from these questions the writer is rather doubtful as to whether strain-gauge measurements on lattice-bars really give an accurate indication of the shear force in columns. The results obtained are quite variable, and are undoubtedly influenced by the flexure of these thin bars. Nevertheless, they are of service in that they confirm the evidence that the shear is small, and that for axial loading the usual assumption of from 2 to 2½% of the longitudinal load is approximately correct.

The writer is in thorough agreement with the Committee's proposals relative to future work. He conceives that the proposed investigations under both Part (1) and Part (2) will lead to just the kind of information that is lacking, and, in particular, is most pleased that it is proposed to investigate the action of actual columns in trusses. Such observations, he is confident, will lead to the establishment of a really rational basis for the design of columns.

J. H. GRIFFITH,[59] M. AM. SOC. C. E. (by letter).—This report presents the data of a carefully conducted series of tests which clarify many points not fully understood in steel column behavior. Among the more important subjects considered, may be mentioned: The effects of the end conditions and eccentric placements of columns in the testing machine; the weights of influence to be attached to irregular deportments of lattice-bars, such as the pantograph action; the proper strengths of lattice and batten-plates; cross-sectional variations; and unintentional cambering of columns. Supplementing its tests, the Committee gives an extended review of important column tests made in the past. This aids much in crystallizing thought as to what constitutes the highest efficiency in column design and fabrication.

---

[59] Prof. of Experimental Eng., Iowa State Coll., Ames, Iowa.

Among the various devices discussed, the frictionless end-bearing will commend itself to testing experts. It calls to mind the earlier forms of bearings used by Bauschinger, von Tetmajer, and Considére,[60] with which the more modern bearing permits favorable comparison. The new bearing seems very substantial and quite adequate for the heavier column loadings of present-day practice in testing. Although no provision is made for the possibility of some torsion in the columns, which is taken into account in the higher analysis, actual bridge columns in service are largely constrained to displacements in the plane of the truss without twist. A review of the test data shows that these bearings fulfilled the intents of the experimenters and were sensitive enough to respond to very small variations in the intentional eccentricities used in the tests. The determinations of the frictions at the column ends to an order of magnitude of a single percentage seem to be amply sufficient. These findings help to remove one of the chief uncertainties present in column investigations—the indetermination existing at the terminals.

The apparatus used for the oblique loading tests is unique and quite simple in construction. Tests of the kind submitted should be appreciated by architectural and mill engineers who, of necessity, must load small columns with extreme eccentricities, as for example, the columns used to support the light crane runways of machine shops. The specimens tested in oblique loadings showed the irregular and discontinuous kind of failures expected in the case of batten-plate and lattice-bar constructions in some instances; in others, they showed the familiar reverse curves for elastic lines as described in various treatises.

Some discrepancy arises between the theoretical derivations and the actual results experienced when latticed columns are tested transversely. This shows one type of inefficiency, among the many others, which occurs in lattice-bar constructions. The method of using bubble levels commends itself in measuring the slopes of the elastic lines for columns in service that are tested in conformity with Manderla's analysis for secondary stresses. This method has been used in Germany to some extent and should be accorded a wider usage because it gives a direct measurement of the variation in curvature which is the cause of the strain usually determined by extensometers or strain-gauges.

The apparatus used for obtaining the stresses in lattice-bars, which are treated as small columns individually, appears to secure results that simulate in behavior (although in a magnified form) the type of strain which will actually take place in the column. It is easily constructed and applied to the ordinary testing-machine platens. Until engineers design a more efficient form of transverse bracing than the lattice-bar type with its many anomalies and enigmas of deportment, it seems desirable to make as many investigations as possible, so that the material supplied for the purpose may be ample. If once a device is used, however unsatisfactory, it tends to persist by a kind of inertia and is very difficult to displace however conscious most engineers may be of the numerous defects, such as those in lattice construction.

---

[60] Discussed by the late J. B. Johnson, M. Am. Soc. C. E., in the earlier editions of his book, "The Materials of Construction".

The analyses submitted and the remarks pertaining to the integral behaviors of columns exposed to strain are usually of most interest to engineers. The question naturally arises, in the light of what has been reported by the Committee, as to what is the most approved form of column equation? Furthermore, does the formula actually check with the test results? Is the formula completely determinate and what details best realize integral behavior? The Committee is apparently inclined to the opinion that the well-known secant formula is the most useful and determinate when the necessary data are obtainable. This was shown quite clearly in the H-sections, the agreement between calculations and tests being all that could be expected, considering the numerous variables that are really under consideration.

A fair confirmation between theory and tests was found in the case of some large bridge columns of alloy steel reported by the U. S. Bureau of Standards.[61] When the results were interpreted according to methods suggested by Henry S. Prichard, M. Am. Soc. C. E., and Dr. O. H. Basquin, some very interesting agreements were found relative to predicted and actual results. The large columns tested followed the secant law until the pin-plates slipped or until the end plane of the column ceased to rotate about its peripheral edge of contact with the upper testing machine platen so that the joint closed. Then, because the eccentricity became practically constant, the stress was relatively constant. This phenomenon was occasioned by the difficulty of getting a uniform contact with the platen and would undoubtedly have disappeared if a roller bearing of the type used by the Committee had then been available. The reason that the stress-strain diagrams in such cases are slightly curved instead of being straight lines is because the assumed mathematical rotation about the outer edge is, in fact, a rocking which is probably caused by the line contact becoming blunted under the column pressure resulting in a slight changing of the eccentricity taking place under stress.

The secant column formula is, in the writer's opinion, the logical consummation of the Bernouilli-Euler or elastic line analysis as applied to compression members. When the equation is developed in the series form by expanding the secant expression, it shows, by a term-to-term comparison (neglecting appropriate terms of higher order), the degree of rationality that attaches to the various short formulas that have been suggested, namely, the short-column formula, the Johnson "straight-line" formula, the parabolic formula, and, as a limiting case, the well-known Euler expression for very slender columns. While the Rankine formula cannot be thus included it is an easy matter to show the type of discrepancy that occurs. The writer believes that if the secant formula were developed in elliptical or Weierstrass sigma functions as was once suggested by Dr. George R. Olshausen in correspondence at the time of tests of the large bridge columns previously mentioned, such an extension would represent the Bernouilli-Euler elastic line analysis to perfection. It would include all the other shorter formulas based on the elastic line analysis to the extent in which these short formulas are admissible as logical derivations according to the law of expressing the bending moment as a function of the curvature. It would serve as the norm of comparison for elastic line column analyses as fast as careful analyses were presented.

---

[61] *Technologic Paper No. 101,* U. S. Bureau of Standards.

In its summary the Committee re-affirms the opinion, advanced from time to time in earlier papers, that the tensile elastic limits of test specimens give the criteria for determining the compression strengths for columns of moderate slenderness ratios. This rule was followed with satisfactory results by most of the consulting engineers, who observed the Bureau of Standards tests, in their estimates of the strengths. Not merely the elastic limits and yield stresses, but the maximum and ultimate stresses, are really conditioned by the rates of loading; therefore, the time factor should enter column analyses as a parameter. In making calculations of the stresses and strains in test pieces, the speed of the straining heads on the testing machine (from 0.05 in. to 12 in. per min.) should be taken into account. The strength of a column under test is likewise affected by the speed of the straining head. The apparent strength may be augmented by increasing the speed.

Items 19 and 20 of the Committee's "Summary of Results" call attention to the difficulties of securing integral action in the column and an appropriate distribution of stresses as the result of the presence of lattice-bars. As the bar systems shorten under pressure caused by pantograph action, they force the column ribs to deflect laterally, which is a behavior not contemplated in efficient design. To be sure the intermediate bars introduced by the late C. H. Cartlidge, M. Am. Soc. C. E., on the test columns for the Metropolis Bridge,[62] eliminated this entirely and did much to insure integral action. Nevertheless, there are compressions introduced in the lattice-bars, as mentioned by the Committee, together with unsatisfactory secondary stresses and alternating actions under stress, that cannot very well be avoided. Since the writer believes that lattice-bars appear to have more disadvantages than advantages, it is pertinent to ask why they should be retained in column construction. Mr. James E. Howard once showed[63] that, under normal conditions, they serve no function in taking stress.

It would seem to the writer that lattice-bars and their attendant evils may be easily omitted altogether: (1) By the use of transverse diaphragms or cross-braces spaced at intervals equal to about twice the depth of the member; and (2) by a suitable reinforcement of the rib flanges in the plane of the truss, not only to maintain a properly balanced section about the column centroid but also to carry any shear normal to this plane. On the side of the column where the lattice-bars are to be replaced there should be a transverse strut of the type used by Mr. Cartlidge[62] between lattice panels in maintaining the integral behavior of the column. Such transverse diaphragms and struts would effectively stiffen the column and reinforce it against the possibilities of twisting. It would insure unity of action in the manner evidently intended by the late Mr. Cartlidge, thoroughly holding the open face flanges in line. The construction would be freer of access for inspection and painting and the diaphragms could be made ample to carry any normal shear, such as that due to heavy wind pressure. There would be none of the anomalous behaviors that are now encountered in the case of lattice-bars. Extra longitudinal ribs or diaphragms could be added if desirable. While the diaphragms would introduce a greater efficiency of behavior, the writer believes the cost of construction would be less.

---

[62] *Technologic Paper No. 101*, U. S. Bureau of Standards.

[63] *Transactions*, Am. Soc. C. E., Vol. LXXIII (1911), p. 492.

The two suggestions under the paragraph "Suggestions Relative to Future Work" are very pertinent. It is very important to insure a continuity of action within the steel as well as in the components and details. Since, on the large alloy-steel columns previously mentioned, all the pin plates slipped and there was evidence of considerable mechanical hysteresis, it might be well to consider the possibility of welding the pin plates and other component parts in addition to the riveting of columns. In the best class of work, the present riveting and reaming practice should be continued to insure continuity and unity of behavior. The costs and labor saved by the omission of lattice-bars, etc., could be applied to the extra expense incurred by electro-welding. Moreover, it is a maxim in the science of dynamics, first enunciated by Kelvin[64] and later re-affirmed by Lamb,[64] that the irrotational motion of a liquid, within a region having a single boundary, is accompanied by the storing of less energy consistent with the same normal motion of the boundary. The late George F. Swain, Past-President, Am. Soc. C. E., and Mr. Frank Cilley long ago stated[65] the closely analogous proposition that structures with the fewest statically indeterminate component parts are not only the most efficient carriers of stress, but that they are the most economical to construct. This fact is recognized in most conservative specifications.

The writer believes that the time has come when the elastic line analysis for bridge columns may be very favorably superseded by so-called "field methods". When structures are composed of members that are "trunk-line" carriers of stress, without a multiplicity of small statically indeterminate members, it is easy to determine the types of fields by means of the method much used and well illustrated by Bach.[66] He divided a number of his specimen faces into fields of small squares or rectangles which, by their distortions under strain, showed the types of strain occurring in the pieces. In like manner the faces of the column ribs and diaphragms may be similarly divided into small squares or rectangles. In fact, this was done to some extent on the alloy-steel columns tested by the U. S. Bureau of Standards, but the relative inelastic displacements under stress largely rendered the efforts inoperative, causing large sets and hysteresis. Love[67] proves the theorem that, when the surface displacements of the surface tractions are measurable, the problem is uniquely determinate.

---

[64] "Hydrodynamics," Third Edition, p. 45, and footnotes.

[65] *Massachusetts Institute Technology Quarterly*, Vol. 10, 1897, No. 2, p. 267.

[66] "Elasticität und Festigheit."

The proposed method is one which readily adapts itself to the strain-gauge measurements used extensively by A. H. Fuller, H. C. Berry, George E. Beggs, Members, Am. Soc. C. E., and Professor H. L. Whittemore. The displacement functions, the components of which (measured in the direction of the co-ordinate axes, say, x, y, and z), are usually designated as u, v, and w, may be found by summing the respective elongations of the small squares, for example, of sides 8 or 10 in. long. The strains, expressed as the partial derivatives, $\frac{d\,u}{d\,x}$, $\frac{d\,v}{d\,y}$, and $\frac{d\,w}{d\,z}$, etc., are found by differentiating the surfaces: $u = u\ (x, y, z)$; $v = v\ (x, y, z)$; and $w = w\ (x, y, z)$. Having found the strains or displacements, it is a simple matter of differentiation to obtain the stresses expressed as functions of the position variables, x, y, and z, referred to a suitable origin on the column faces. Numerous examples of procedure are given by Love[67] and by Prescott[68] in a manner more suitable for the use of engineers. Attention is called to interesting plats of the surfaces obtained by similar methods herein proposed on a celluloid model of the Stevenson Creek Dam by Professor Beggs[69] and by W. A. Slater, M. Am. Soc. C. E.[70] and to the type of analysis to be used by H. M. Westergaard, M. Am. Soc. C. E., in his analyses of the same arch dam.[71]

The column analyses may be simplified if the stress equations are suggested by the secant type of formula, or by Manderla's extension of the elastic line theory as applied to trusses, the latter affording the cue to the class of functions desired. The writer has applied these methods with excellent results to complicated problems in earth resistance, in which large strains and hysteresis phenomena are present. The problems of the column are simplified by eliminating strain hysteresis in the manner already discussed. Moreover, one need only consider the first-order strains of the theory of elasticity in deriving the surfaces.[72] The addition of many small members will, of course, render such methods practically prohibitive by complication of the processes.

---

[67] "Theory of Elasticity," Second Edition, p. 167.

[68] "Applied Elasticity."

[69] *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 224.

[70] *Loc. cit.*, pp. 133, 135, *et al.*

[71] *Loc. cit.*, p. 221 *et seq.*

[72] See treatment of simple surfaces by writer in "Proportioning of Pit-Run Aggregates for Concrete," *Engineering News-Record*, March 10, 1927, p. 399; also, in "Pressures under Substructures," *Engineering and Contracting*, March, 1929.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1790

SEWAGE WORKS OPERATION AND CONTROL

FINAL REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION[1]

This Committee was appointed in 1926 by the Executive Committee of the Sanitary Engineering Division, to deal with the underlying principles the adoption of which seem to insure the satisfactory operation and control of sewage works. A brief progress report was made by the Committee and presented to the Division at the Annual Meeting of the Society on January 19, 1928.[2] Since that time the Committee has been studying such questions as the best manner of obtaining funds to operate sewage works, the qualifications and adequate compensation of the personnel employed in such works, the establishment of schools for sewage works operators, the question of control analyses at sewage works, etc.

The Final Report of the Committee was presented to the Executive Committee of the Sanitary Engineering Division on April 9, 1929. After thorough discussion, it was adopted by the Division at its meeting in Cleveland, Ohio, on July 10, 1930. In the light of this action and after further review, the report is herewith presented in its final form.

C. E. Keefer, *Chairman*,
W. R. Copeland,
A. L. Fales,
J. A. Vogleson,
F. H. Waring,
June 17, 1931. *Committee*.

The Operation and Control of Sewage Works

The art of designing and constructing sewage works has advanced to the stage where municipal sewage can be treated adequately in order to produce an effluent of any reasonable degree of purity. The operation of such works, however, has not kept pace with the advances made in their

---

[1] Presented at the meeting of the Sanitary Engineering Division, Cleveland, Ohio, July 10, 1930.

[2] *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 153.

design. A survey of these works has indicated that many of them are poorly operated, with the result that the welfare and health of many communities are imperiled. The outstanding reasons for the unsatisfactory operation of sewage treatment plants are the lack of sufficient operating funds and the need of well-trained and efficient personnel. The purpose of this report is to outline the underlying principles which it seems necessary to follow to insure the satisfactory operation of sewage works.

STATE SUPERVISION OF SEWAGE WORKS

Each State department of health or other similar State board through adequate legal statutes should have approval of the design and control of operation of the sewage works in its own State. The State board should also have sufficient appropriations and adequate personnel to keep in touch with all sewage treatment works in the State. In case a sewage plant is not operated and maintained in a satisfactory manner the board should have the legal right to require the owner to employ a competent person or persons to operate the plant and make such repairs and improvements as may be necessary. The necessary legislation to accomplish this purpose is typified by the Ohio law given in Appendix I. Where adequate laws do not exist, legislation should be passed conferring such powers upon the State board of health.

FINANCING SEWAGE WORKS OPERATION

One of the most important matters affecting the successful operation of sewage works relates to the securing of sufficient funds for this purpose. Most sewage works owned and operated by municipalities, or other political sub-divisions, are financed out of the general tax levies. It is frequently the aim of those public officials who decide the amount of money to be appropriated for annual operating budgets to keep the tax rate as low as possible. The result is that funds for the efficient operation of sewage plants are generally inadequate. Such have been the findings from a survey made by the Committee. In order to correct such a condition attempts have been made to find some other practicable method of financing sewage works operation.

The Legislature of the State of Ohio has passed a law (see Appendix II) which permits the council of any city or village in that State to establish by ordinance equitable rates or charges for sewerage service. It is provided in the Ohio law that these rates are to be paid only by those persons who are served by the sewerage system. In Ohio a number of the smaller cities, including Delaware, Oxford, and Salem, and the large city of Dayton, have passed the necessary ordinances as a result of this law. The ordinances providing for sewer rentals for Delaware, Dayton, and Oxford, are given in Appendix III.

The Committee believes that as far as practicable the maintenance and operation of all sewage works as well as the tributary sewerage systems should be financed by funds collected from the persons or property owners who are benefited. These funds should be levied in the form of rents, payable at stated intervals, and should be used only for the maintenance and operation of the sewage works and the sewerage system. The financing of the cost of construction of sewerage works should be borne by the entire communities benefited.

QUALIFICATIONS FOR SEWAGE WORKS SUPERINTENDENTS

The operation of sewage works should be in the hands of competent men. The number of the personnel and their training depends upon the size and the type of the treatment plant. The responsibility for the satisfactory operation of the plant rests largely with the superintendent. For that reason he should have the necessary training and experience to qualify him properly for the responsibilities of his position.

The qualifications for superintendents of sewage works of different types and sizes have been classified roughly, as follows:

A Grade A Superintendent should be a graduate of an engineering or technical school of recognized standing, and should have had experience for not less than five years in problems relating to sewage treatment. He should also be able to supervise and direct the work of others.

A Grade B Superintendent should be at least a high school graduate with additional training in chemistry and engineering, and should have had experience for not less than five years in problems relating to sewage treatment. He should be able to supervise and direct the work of others.

A Grade C Superintendent should have had at least an elementary school education with the training equivalent to that of a skilled mechanic or skilled laborer. He should be able to supervise and direct the work of others.

Grades of Superintendents Required.—All sewage treatment plants should be operated under technical control or supervision.

In the case of large plants, serving approximately 100 000 or more people and involving such treatment as by Imhoff tanks, sedimentation and separate sludge digestion, trickling filters, or the activated sludge process, Grade A Superintendents should be employed.

For moderate-sized plants, serving from approximately 50 000 to 100 000 people, Grade B Superintendents may be satisfactory; but for the more complicated plants it is desirable to have the operation supervised by a sanitary engineer or chemist who has the qualifications of a Grade A Superintendent.

In the case of less complicated plants, such as fine screening, serving less than about 50 000 persons, Grade C Superintendents may be satisfactory, provided there is adequate supervision by a sanitary engineer or chemist with the qualifications of a Grade B or a Grade A Superintendent. For small plants of a complicated type, or for dealing with difficult conditions, a Grade B or Grade A Superintendent may be required.

Compensation for Sewage Works Superintendents.—In order to attract and secure properly qualified men to act as superintendents their compensation should be commensurate with their responsibilities. The Committee suggests the following relative salaries, based on common labor receiving a wage equivalent to 1.0:

Grade C Superintendent	1.0 to 2.0
Grade B Superintendent	2.0 to 3.0
Grade A Superintendent	3.0 to 5.0

This method of rating has been adopted because the scale of wages is not uniform in all places, and varies from time to time in the same locality.

The practice followed by some communities of having a competent sanitary engineer to advise the officials regarding problems of sewage works operation is a good one. This plan is especially desirable for a period immediately following the opening of a new sewage works. Where a community cannot afford to pay for the services of a full-time technically trained superintendent, it may be possible to unite with near-by communities in employing an experienced chemist or sanitary engineer for the supervision of operation, or it may be possible for a small municipality to secure the part-time services of a technically trained superintendent of some large sewage plant to supervise the operation.

Schools for Sewage Works Operators

Many of the operators and other assistants at sewage works, particularly at the small plants, lack the basic training which is necessary to understand the fundamentals of sewage treatment and to supervise and operate in a satisfactory manner the plants they control. A number of State departments of health are awake to this condition, and as a corrective measure there have been established in some States short-time schools, where instruction in the general principles of sewage treatment is given.

The need for this type of training is so great that additional schools should be provided. It would be desirable to have at least one school for each State, except where the number of sewage works is small, in which case it may be feasible to organize one school for two or more adjacent States. The various factors involved in conducting these schools will depend upon local conditions. The most important points to consider are the organizing and financing of the school, the character of instruction to be given, and the paying of the students' expenses.

State departments of health and sewage works associations are the logical organizations to establish and promote schools for sewage works operators. Where there is a suitable State educational institution conducting extension work, this is the most desirable place to conduct the school. As such institutions are financed by public funds only nominal fees would be charged. Students would thus have no great difficulty in paying for their instruction.

The general purpose of the course should be to train students in the fundamentals of sewage treatment and in making laboratory tests. The instruction should include lectures, laboratory work, and inspection trips. The lectures should deal with elementary sewage plant design, and the fundamentals of hydraulics, chemistry, and bacteriology. The scope of the laboratory work should vary with the length of the course and the previous training of the student. The most elementary course should include the keeping of records, the collection of samples, the measuring of settleable solids with Imhoff cones, the relative stability test, the determination of the pH of sewage and sludge, and the operation of chlorine apparatus. These studies may be supplemented, if time is available, or if an advanced course is given, by practice in the use of the chemical balance, the determination of suspended solids, dissolved oxygen, bio-chemical oxygen demand, and the numbers of bacteria and *B. coli*. The length of the course would have to be limited to a short period—preferably not less than one week. A two-week session would possibly be the maximum length of time that could be devoted to this work.

The method of financing the students' tuition and expenses will depend upon local conditions. In those municipalities where the officials are convinced of the importance of such schools it is possible that the major part, or perhaps all, of the expense will be borne by the municipality. Legal limitations governing the expenditure of municipal funds may be a controlling factor. In any event, the training received by the operator should increase his capabilities and add to the value of his services. The Committee recommends that the necessary expenses of sewage works operators for attending schools or conferences be paid by the municipalities.

Sewage Works Associations

Membership in sewage works associations affords another means of acquiring valuable knowledge relative to the operation of sewage treatment plants. Several of these associations have been formed in a number of States or groups of States. These associations usually meet at least once a year for a two or three-day convention when papers are presented, practical problems are discussed, and inspections are made of plants of interest.

A Federation of Sewage Works Associations has been formed, which publishes a *Sewage Works Journal*. This publication affords the sewage works operator an additional opportunity of familiarizing himself with problems relating to sewage treatment.

Some State departments of health are responsible for calling conferences of sewage works operators for the purpose of promoting an exchange of ideas regarding sewage treatment problems and for improvement in economy and efficiency of the operation of sewage works within the State.

Analytical Determinations and Tests Required at Sewage Works

There are three main objects of making analyses and tests at sewage treatment works: (1) For control of operation; (2) for record of accomplishment; and (3) for research investigations.

The number and kind of analytical determinations and tests to be made will vary with the size and type of works. For small plants the routine tests may be confined to those required for the control of operation. For larger plants, especially where litigation may ensue, it is important to do more laboratory work than is required for the control of operation alone. Data should be obtained, not only for studying the efficiency of the plant, but also for comparison with similar data from other plants. At large sewage works, particularly where there are problems involving improvements or enlargements to the plant, or the recovery of by-products, research work should be carried on.

Research in the treatment of sewage and industrial wastes is greatly needed, not only to promote economy and efficiency of treatment, but also to determine ways of recovering valuable by-products. The analytical determinations and tests involved in research investigations are too varied to designate in this report, but will depend upon the problem being studied.

The analytical determinations and tests required for the control of operation and for a record of accomplishment may be classified according to the treatment provided. It is impracticable in this report to cover all kinds of sewage treatment, but some of the more widely used methods will serve for illustrative purposes.

Sewage Flow and Population Served.—It is very important that a continuous record of the sewage flow be kept. Where Venturi meters have not been installed, the use of weirs, or some other means, should be resorted to, in order to determine the sewage flow. The quantity and time of by-passing sewage around all or any part of the sewage works, should be noted. During storms, the volume of storm-water, which does not receive treatment, should be recorded. Where an appreciable quantity of industrial wastes is treated, the volume, character, and equivalent population load should be determined.

Screening.—For the control of the operation of screens, the quantity of sewage screened and the volume of screenings obtained should be determined. The time that the screens are in service, the work required on them, and their power consumption should be noted.

For a record of accomplishment the screenings, where possible, should be weighed, and the water content determined. The amount of solids removed from the sewage can then be computed. The efficiency of screening in removing suspended solids is accurately determined by adding to the weight of suspended solids determined in the screen effluent the weight of dry solids in the screenings from a corresponding volume of sewage, and computing the percentage of solids removed by the screens. By determining the loss on ignition of the suspended solids in the effluent and of the solids in the screenings, the percentage removal of organic matter can be estimated in a similar manner.

Sedimentation.—A minimum requirement for the control of treatment by sedimentation should be the conical measuring glass test for the volume of settleable solids in the influent and effluent. The quantity of sewage and the volume of accumulated sludge are also important factors in the control of the treatment.

For a record of accomplishment the removal of suspended solids, by weight, should be ascertained by gravimetric determination of suspended solids in the influent and in the effluent. It is also important to determine the removal of organic matter. This may be done by determining the organic nitrogen or albuminoid nitrogen and oxygen consumed in the influent and the effluent. The reduction in bio-chemical oxygen demand has come to be considered an important index of the purification accomplished. Tests for dissolved oxygen in the influent and the effluent should be made with sufficient frequency to show whether the sewage has become septic.

Trickling Filter Treatment.—Relative stability tests of the effluent are desirable for the control of operation of trickling filters. The volumetric determination of settleable solids in the sewage applied and in the effluent is an index of the extent of the storage or unloading of solids in the filters. The volume of sewage treated by the filters and the condition of the filters with respect to clogging are also important factors in the control of operation.

For a record of accomplishment the reduction in organic matter can be ascertained by determining the organic nitrogen or albuminoid nitrogen and oxygen consumed, or of bio-chemical oxygen demand in the influent and in the effluent. The extent of oxidation is further shown by reduction in free ammonia nitrogen and increase in nitrogen as nitrites and nitrates.

Where trickling filter effluents are passed through sedimentation tanks, the same determinations should be made of the tank effluent as have been recommended for the filter effluent. The suspended solids in the trickling filter effluent and the tank effluent should be determined gravimetrically. The dissolved oxygen in the effluent before and after sedimentation will indicate, not only the degree of oxidation, but also the effect of the sludge in the sedimentation tank.

Activated Sludge Treatment.—For the control of activated sludge treatment, the flow of sewage, the proportion of activated sludge returned to the sewage, the volume of air applied, and the amount of power used, are important factors. The proportion of activated sludge present can be determined by sedimentation tests or more quickly by the centrifuge. A knowledge of the amount of solids in the sludge by weight is also important in the control of treatment. Tests for turbidity, suspended matter, and relative stability should be made on the effluent after sedimentation. The sludge level in the sedimentation tanks should be kept under control by measuring the depth of sludge in the tanks and by observing the character of the sludge withdrawn.

For a record of accomplishment, in addition to these tests, determinations of total nitrogen or albuminoid nitrogen and oxygen consumed, or of bio-chemical oxygen demand, should be made on the sewage and the effluent. The extent of oxidation is indicated by determinations of free ammonia nitrogen and nitrogen as nitrates, and dissolved oxygen in the effluent.

Disinfection.—Tests for the control of disinfection should be made for residual chlorine after an adequate period of contact. For a record of accomplishment bacteria counts and *B. coli* tests should be made before and after disinfection. In some cases, it will be desirable to determine the reduction in bio-chemical oxygen demand accomplished by disinfection. The quantity of chlorine applied should be noted. For reasons of economy it may be desirable to vary the dosage of chlorine to correspond with the chlorine demand of the sewage.

Sludge Digestion.—For the control of sludge digestion the appearance and odor of the sludge and the reaction, or pH index, during digestion are important. Temperature is also important especially where control is exercised over it. For a record of accomplishment the total solids and organic matter by loss on ignition should be determined of the raw and the digested sludge in order to ascertain the proper volume to withdraw. Where gas is collected, it is desirable to obtain its volume and composition. A record should be kept of the volume and character of the sludge liquor discharged from separate sludge digestion tanks.

Dewatering of Sludge on Sludge Beds.—When sludge is dewatered on sludge beds, the volume of sludge applied, depth of application, time of drying, number of applications per year, weather conditions, and volume of dewatered sludge should be determined. Records of sludge-bed operation should include the percentage of solids in the sludge as applied and as removed.

Mechanical Dewatering of Sludge.—Where sludge is dewatered by mechanical means, it is important to make tests of sludge density, or solid content, and reaction, or pH value, checked by rate-of-filtration tests in connection with the conditioning of the sludge for dewatering. The rate of dewatering by filtration of the sludge and the yield of cake are important. Determinations of the water and the solids in the dewatered sludge should also be made.

When sludge is dried and prepared for fertilizer, the determination of its mechanical composition, percentage of moisture, and of the total and available nitrogen and other fertilizing constituents, is important.

Treatment of Sewage Containing Industrial Wastes.—Where industrial wastes or sewage containing abnormal quantities of industrial wastes are treated, analytical determinations and tests, in addition to those previously mentioned, may be required such as color, fats, alkalinity or acidity, iron, copper, lead, zinc, aluminum, calcium, magnesium, sulfates, and sulfides, depending on the kinds of industrial wastes present.

Effect of Discharge of Sewage or Effluent on the Receiving Waters.—In many cases, especially where there is a possibility of litigation, tests should be made to determine the effect of the discharge of sewage or sewage effluent at different points on the receiving waters. These tests should ordinarily include color, suspended solids, dissolved oxygen, and relative stability or biochemical oxygen demand. The presence of sludge banks should be noted. Periodic determinations of the number and kinds of plankton in the receiving waters may be desirable. In cases where the receiving waters are used for water supply, bathing, or shell fish culture, bacteria counts and *B. coli* tests should be made. The time, extent, and frequency of such tests will depend upon local conditions. In certain instances, particularly in tidal waters and lakes, studies should be made of the currents where sewage effluents are discharged. A knowledge is desirable of the dilution afforded by the receiving waters, particularly during the summer when temperatures are high and stream flows are at a minimum.

Importance of Sampling.—For comparative and analytical determinations and tests it is of the utmost importance that the samples be truly representative. In order to obtain representative composite samples individual samples should be taken at frequent intervals and in proportion to the sewage flow. Determinations of chlorine in the form of chlorides are of assistance in indicating to what extent samples are comparable.

Laboratories.—In order to perform the necessary analytical determinations and tests, it is essential that suitably equipped laboratories should be provided. The larger plants will require complete equipment for making analytical determinations and tests needed for research investigations, as well as for a record of accomplishment. In the case of small plants, it may be sufficient to provide equipment only for the tests required to control the operation. Where local laboratories are not available, control samples should be collected at regular intervals and analyzed by the technical supervisor.

Influence of Local Conditions.—It is impossible to stipulate just what analytical determinations and tests should be made at every sewage treatment plant. Local conditions will often govern. The analytical determinations and tests suggested in this report may be considered generally applicable. Other determinations and tests will be required in special cases. In all instances it is desirable to record the temperatures of the sewage, effluents, and sludges, in order to compare the results in different localities. Sufficiently complete analyses of the sewage should be made to determine its general character and composition.

It is unnecessary in this report to describe the methods of analysis which should be used. Technically trained men, having the control or supervision of the treatment plants, will know the methods of analysis and how to interpret the results.

Operation and Maintenance Cost Data

In addition to complete records of operating and analytical data, operation and maintenance costs should be kept at sewage treatment works. These costs should be segregated into labor and materials. Moreover, it is desirable to keep operation and maintenance costs separate, with a further record of the cost of all new additions or improvements. Should any money be collected from the sale of screenings, sludge, or other materials, the amounts should be recorded. At the large plants it is desirable to make a further segregation by keeping separate accounts of the cost of screening, sedimentation, sludge treatment, etc. A record should be made of the sewage flow and the population served so that costs of treatment per unit volume of sewage and per capita can be calculated.

As improvements are made to the works from time to time and as old units are abandoned, the cost of these should be added to or deducted from the first cost of the plant so that the fixed cost, comprising the interest on the investment and depreciation can be determined.

REPORTS COVERING SEWAGE WORKS OPERATION

In spite of the large number of sewage works of considerable size in the United States, the number of these is remarkably small, at which printed reports giving operating data are prepared. These reports are of much interest and value to all those engaged in the design, construction, or operation of sewage works. It is mutually helpful for sewage works superintendents to exchange reports. It is, therefore, highly desirable that printed reports be prepared, preferably annually, giving the operating and cost data of sewage treatment works, particularly in the case of works of any magnitude. The yearly budget, which provides for the cost of operation, should include an item covering the printing of an annual report. It is also very desirable that articles relating to sewage works operation be published in engineering magazines or journals.

APPENDIX I

OHIO STATE DEPARTMENT OF HEALTH LAW[3]

Sec. 1255. (Order for improvement of water purification or sewage treatment works.) When the commissioner of health finds upon investigation, that any water purification or sewage treatment works, on account of incompetent supervision or inefficient operation is not producing an effluent of such quality as might be reasonably obtained from such water purification or sewage treatment works, and by reason of such neglect the public water supply has become impure and dangerous to health, or that a stream, water-course, canal, lake, pond or body of water has become offensively polluted or has become a public nuisance or that a public water supply taken from such stream, water-course, canal, lake, pond or body of water has been rendered impure and dangerous to health, the commissioner of health shall issue an order to the mayor or managing officer or officers of the city, village, public institution, or corporation, partnership or person having charge of or owning such water purification or sewage treatment works, to secure an effluent of such quality as might be reasonably expected from such works and satisfactory to the commissioner of health. (108 Pt. I, 297.)

Sec. 1256. (Order to appoint and pay competent person; approval by commissioner.) If the managing officer or officers of such city, village, public institutions or corporation, partnership or person fails, for a period of five days after receiving such order, to secure an effluent satisfactory to the commissioner of health, the commissioner of health shall report the fact to the public health council and upon its approval may order such managing officer or officers or person owning such works to appoint within ten days, and pay the salary of a competent person to be approved by the commissioner of health, to take charge of and operate such works as to secure the results demanded by the commissioner of health. (108 Pt. I, 297.)

---

[3] The Ohio law has been sustained on several occasions by the Supreme Court of Ohio.

Sec. 1257. (Right of appeal; procedure.) If the findings or order of the commissioner of health, when approved by the public health council and made in pursuance of the provisions of this chapter relating to stream pollution and public water supply, are not acceptable to any city, village, public institution, corporation or owner affected thereby, such city, village, public institution, corporation or owner shall have the right of appeal as follows: Two reputable and experienced sanitary engineers shall be chosen, one by the city, village, public institution, corporation or owner and the other by the commissioner of health, who shall not be a regular employee of the State department of health. Such persons shall act as referees. If the referees so chosen are unable to agree, they shall choose a third engineer of like standing and the vote of the majority shall be final. As soon as such referees are chosen, the commissioners (commissioner) of health shall file with them a certified copy of the complaint and the findings, and order of the State department of health, and it shall be the duty of such referees to investigate the conditions complained of and to determine if such findings are correct and if the order provides a proper remedy for such conditions. The appeal provided for in this section shall be made within thirty days from the date of service of the order upon the mayor or managing officer or officers of the city, village, public institution or corporation or owner, and notice thereof in writing, shall be served upon the commissioner of health by personal service for which there shall be acknowledgment, or sent by registered letter.

Sec. 1258. (Powers of referees; fees and expenses, how paid.) Such referees may affirm or reject the findings or order of the commissioner of health or may modify such order as to the time within which improvements or changes shall be made, and their decision, which must be in writing and be made within a reasonable time, shall be reported to the commissioner of health and to the city, village, public institution, corporation or owner and shall be final except as hereinafter provided. If said findings and order shall be approved or modified by said referees, the order shall be enforced by the commissioner of health in the manner provided for in this chapter. The fees and expenses of the referee appointed by the commissioner of health shall be paid from funds appropriated to the State department of health for such purpose. The fees and expenses of the referee appointed by the city, village, public institution, corporation or owner shall be paid by the city, village, public institution, corporation or owner making such appeal. The fees and expenses of the third referee shall be equally divided between the State department of health and the city, village, public institution, corporation or owner making appeal. (108 Pt. I, 297.)

Sec. 1258-2. (Vacating or modifying order.) An order as made by the commissioner of health or as approved or modified by the referees as herein provided, shall be reversed, vacated or modified by the Supreme Court on a petition of error, if upon consideration of the record such Court is of the opinion that such order was unlawful and unreasonable. (108 Pt. I, 297.)

Sec. 1258-6. (Supreme Court only has power to suspend order.) No Court other than the Supreme Court shall have the power to review, suspend or delay any order of the commissioner of health, or enjoin, restrain or interfere with the commissioner of health or public health council in the performance of official duties required or power exercised under the provisions of this act. (108 Pt. I, 297.)

Sec. 1260. (Forfeiture for failure to obey orders.) If a council, department or officer of a municipality, or person, partnership or private corporation fails or refuses for a period of thirty days, after notice given him or them by the commissioner of health of his findings and order, and the approval thereof by the public health council, to perform any act or acts required of him or them by this chapter relating to stream pollution and public water supply, the members of such council or department, or such officer or officers, person, partnership or private corporation shall be personally liable for such default, and shall forfeit and pay to the State of Ohio five hundred dollars to be paid into the State treasury to the credit of the general revenue fund. (108 Pt. I, 297.)

Sec. 1261. (Action for recovery.) An action may be begun for the recovery of such penalty by the prosecuting attorney of a county in the name of the State in the Court of Common Pleas of such county having jurisdiction of any such party or parties, or it may be begun by the attorney general in such county or the County of Franklin, as provided by law. The Court of Common Pleas, upon good cause shown, may, at its discretion, remit such penalty or any part thereof. (108 Pt. I, 297.)

APPENDIX II

Ohio Sewer Rental Law[4]

Council may by ordinances establish rates or charges of rent to be paid city or village.

Section 3891-1. The council of any city or village which has installed or is installing sewerage, a system of sewerage, sewage pumping works, or sewage treatment or disposal works for public use, may by ordinance establish just and equitable rates or charges of rents to be paid to such city or village for the use of such sewerage, a system of sewerage, sewage pumping works or sewage treatment or disposal works by every person, firm or corporation whose premises are served by a connection to such sewerage, system of sewerage, sewage pumping works or sewage treatment or disposal works. Such charges shall constitute a lien upon the property served by such connection and if not paid when due shall be collected in the same manner as other city and village taxes. The council may change such rates or charges from time to time as may be deemed advisable. Provided, however, that in a municipality operating under a municipal charter the council or other legislative body may establish the schedule of rates herein authorized and provide for its administration by designating the department or officer of the municipality to be charged with the enforcement of the provisions of this act.

---

[4] Passed April 5, 1923.

Director of public service, general duties.

Section 3891-2. In a city the director of public service shall manage, conduct and control the sewerage system and sewage pumping, treatment and disposal works, and when the council has established a schedule of rates or charges of rents for their use shall collect sewer rentals, and he shall appoint the necessary officers and agents for such purposes.

Board of trustees of public affairs shall manage and control sewerage system, etc.

Section 3891-3. When the council of a village has established a schedule of rates or charges of rents for the use of the sewerage system and sewerage pumping, treatment and disposal works, the board of trustees of public affairs shall manage, conduct and control such sewerage system and sewage pumping, treatment and disposal works and shall collect sewer rentals and shall appoint necessary officers and agents for such purposes; and shall assume all duties, in relation to sewers, imposed upon the street commissioner by Section 4364 of the General Code, whereupon the street commissioner shall be relieved of such duties.

By-laws and regulations.

Section 3891-4. The director of public service and the board of trustees of public affairs may make such by-laws and regulations as may be deemed necessary for the safe, economical and efficient management and protection of the sewerage system and sewage pumping, treatment and disposal works; and for the construction and use of house sewers and connections to the sewerage system. Such by-laws and regulations shall have the same force and effect as ordinances when not repugnant thereto or to the constitution or laws of the State.

Reception and distribution of funds derived from sewer rentals.

Section 3891-5. The funds received from the collection of sewer rentals shall be deposited weekly with the treasurer of the corporation. Money so deposited shall be kept as a separate and distinct fund and shall be known as the sewer fund. When appropriated by council, it shall be subject to the order of the director of public service of a city or board of trustees of public affairs of a village. Such director or board shall sign all orders drawn on the treasurer of the corporation against such fund. This fund shall be used for the payment of the cost of the management, maintenance, operation and repair of the sewerage system and sewage pumping, treatment and disposal works and any surplus in such fund may be used for the enlargement or replacement of the same and for the payment of the interest on any debt incurred for the construction of such sewerage system and sewage pumping, treatment and disposal works, and for the creation of a sinking fund for the payment of such debt, but shall not be used for the extension of a sewerage system to serve unsewered areas or for any other purpose whatsoever.

Section 3891-6. That Section 4357 of the General Code shall be amended to read as follows:

When board shall be established. Section 4357. In each village in which water works, an electric light plant, artificial or natural gas plant, or other similar public utility is situated, or when council orders water works, an electric light plant, natural or artificial gas plant, or other similar public utility, to be constructed, or to be leased or purchased from any individual, company or corporation, or when the council shall have determined to establish a schedule of rates or charges of rents for use of the sewerage system and sewage pumping, treatment and disposal works of the village, council shall establish at such time a board of trustees of public affairs for the village, which shall consist of three members, residents of the village, who shall be each elected for a term of two years.

APPENDIX III

SEWER RENTAL ORDINANCE NO. 1125, DELAWARE, OHIO[5]

To Provide Funds for the Maintenance, Operation and Control of the Sewerage System and Sewage Treatment Works of the City of Delaware. Ohio:

WHEREAS, The City of Delaware, Ohio has installed a Sewerage System and Sewage Treatment Works, and

WHEREAS, It is necessary for the preservation of the public health and for compliance with orders of the State Department of Health, that the said sewerage system and sewage treatment works be properly maintained, operated and controlled, and

WHEREAS, Funds for the maintenance, operation and control of said sewerage system and sewage treatment works will not be available from the current operating revenue of the City, and

WHEREAS, The General Assembly of the State of Ohio has enacted a law, amending Section 4357 of the General Code, to permit municipalities to assess and collect for the use of sewers, said charges to provide for the proper maintenance, operation and control of sewerage systems and sewage treatment or disposal works,

NOW, THEREFORE, Be it Ordained by the Council of the City of Delaware, State of Ohio:

---

[5] Estimated 1929 population about 9 000 for city, and 2 500 for Ohio Wesleyan University. Passed August 1, 1927.

Section 1. That the Director of Public Service be and he is hereby authorized and directed to manage and control the sewerage system and sewage treatment works in a safe, economical and efficient manner.

Section 2. That the Director of Public Service be and he is hereby authorized and directed to make such by-laws and regulations as may be deemed necessary for the safe, economical and efficient management and protection of the sewerage system and sewage treatment works, and for the construction and use of house sewers and connections to the sewerage system; that such by-laws and regulations shall have the same force and effect as ordinances when not repugnant thereto nor to the constitution or laws of the State of Ohio.

Section 3. That the terms and conditions upon which the owners of lots and lands within the corporate limits of the City of Delaware, may connect to the sewerage system either directly or indirectly, shall be as follows:

Article 1. Charges shall be paid at the office of the Director of Public Service, and for each calendar year or part thereof they shall be in accordance with the following schedule of rates:

Dwellings and Apartments:

One-Family (one connection), $4.00 per annum.
Two-Family (one connection), $7.50 per annum.
Two-Family (two connections), $8.00 per annum.
Four-Family (two connections), $15.00 per annum.
Others (for each family), $4.00 per annum.
For each additional family on the same connection, $3.75 per annum.

Hotels:

For each room with bath, $1.75 per annum.

For each room without bath, $1.00 per annum.

Rooming Houses and Fraternities:

Four Dollars ($4.00) per annum plus One Dollar ($1.00) per annum for each person renting a room therein.

Stores, Restaurants, Offices and Mercantile Establishments:

Private toilets, each $4.00 per annum.
Public toilets, each $8.00 per annum.
Kitchen sinks, each $4.00 per annum.
Slop sinks, each $2.00 per annum.

Factories and Shops:

For each and every person employed or engaged in service on the premises, in accordance with the following schedule:

Ten persons or less, each $1.00 per annum.
More than 10 and not over 25, 90 cents per annum.
More than 25 and not over 50, 80 cents per annum.
More than 50 and not over 100, 70 cents per annum.
All over 100, 60 cents per annum.

Churches.

For each toilet, $2.00 per annum.

Colleges:

All college buildings and dormitories not subject to taxation shall be grouped and the sewer rental fixed on per capita student population:

For each registered student, 75 cents per annum.

Hospitals and Sanitariums:

Delaware Spring Sanitarium, $100.00 per annum.
Jane M. Case Hospital, $60.00 per annum.

Miscellaneous:

For special cases which are not clearly defined in the above schedule, rates shall be established by Council upon the recommendation of the Director of Public Service.

Article 2. Charges shall become due and payable in advance on the first day of January, April, July and October. All bills not paid by the fifteenth of the month in which same becomes due shall be subject to a 10 per cent. penalty.

Article 3. Charges shall constitute a lien upon the property served by such connections, and if not paid within ninety days, shall be collected in the same manner as other city taxes.

Article 4. Charges for discharging industrial wastes into the sewerage system shall be fixed by the Director of Public Service and approved by Council.

Section 4. That the funds received from the use of sewers shall be deposited with the Treasurer of the City, and money so deposited shall be kept in a separate and distinct fund and shall be known as the Sewer Rental Fund. When appropriated by Council it shall be subject to the order of the Director of Public Service and said Director shall sign all orders drawn on the Treasurer of the City against said fund.

Section 5. Funds received from the use of sewers shall be used for the payment of the costs and expenses of the management, maintenance, operation and repair of the sewerage system and sewage treatment works and any surplus in such fund shall be used for the enlargement or replacement of the same or for the payment of the interest on any debt incurred for the construction of such sewerage system and sewage treatment works, or for the creation of a sinking fund for the payment of such debt, but shall not be used for the extension of the sewerage system to serve unsewered areas nor for any purpose whatsoever.

Section 6. That this ordinance constitute a part of an emergency measure and shall take effect and be in force from and after its passage.

SEWER RENTAL ORDINANCE NO. 13448, DAYTON, OHIO[6]

WHEREAS, The City of Dayton has built and has in operation a sanitary sewerage system for the purpose of collecting sanitary sewage and conveying the same away from the premises where produced to a place of disposal, and is constructing intercepting sewers, pumping stations, force mains and a disposal works for the treatment of such sewage; and

---

[6] Estimated 1929 population about 180 000. Passed by the Commission, December 28, 1927.

WHEREAS, The City Commissioner finds, in order to promote the public health, safety, welfare and convenience, that a system of sanitary sewerage, including intercepting sewers, and force mains, sewage pumping works and a sewage treatment works, are necessary, and that in order to pay the cost of the management, maintenance, operation and repair, or for the enlargement and replacement of the sanitary sewerage system and the sewage pumping, treatment and disposal works and to pay the interest upon the bonds issued and outstanding and to be issued to provide funds for the construction thereof and to retire said bonds as they mature, it is necessary to levy and collect a charge of rental upon the lots, lands and premises served by sewerage connection with the sanitary sewer system of said City; therefore,

BE IT ORDAINED by the Commission of the City of Dayton:

Section 1. It is hereby determined and declared to be necessary and conducive to the protection of the public health, safety, welfare and convenience of the City of Dayton, Ohio, to levy and collect charges or rentals upon all lots, lands and premises served by having connections with the sanitary sewerage system and by the sewage, pumping, treatment and disposal works of said city, the proceeds of such charges or rentals so derived to be for the use of the sanitary sewerage system and the pumping, treatment and disposal works of said city, as hereinafter provided.

Section 2. For the purposes of this ordinance the terms, "Sanitary Sewage" and "Industrial Wastes", are defined as follows:

(a) "Sanitary Sewage" is hereby defined as the waste from water closets, urinals, lavatories, sinks, bath tubs, showers, household laundries, cellar floor drains, garage floor drains, bars, soda fountains, cuspidors, refrigerator drips, drinking fountains and stable floor drains.

(b) "Industrial Wastes" are hereby defined as being the liquid waste resulting from any commercial, manufacturing or industrial operations or processes.

Section 3. For the purposes as provided in Section 1 and 8 hereof there is hereby levied and assessed upon each lot, parcel of land, building or premises having any sewer connection with the sanitary sewerage system of said city or otherwise discharging sewerage, industrial wastes, water or other liquids, either directly or indirectly into the City sanitary sewerage system, a sewerage service charge or rental payable as hereinafter provided, and in amount determinable as follows:

(a) For any lot, parcel of land, building or premises, situated within the corporate limits of the City of Dayton, Ohio, and having any connection with the city sanitary sewerage system or otherwise discharging sanitary sewerage, industrial wastes, water or other liquids, either directly or indirectly, into the City sanitary sewerage system, said charge or rental shall be based upon the quantity of water used thereon or therein as the same is measured by the City water meter there in use, and there shall be charged for the first 3 000 cubic feet of water, or part thereof, so used per quarter year, the sum of One Dollar ($1.00); for each thousand cubic feet of water used in excess of 3 000 cubic feet and not exceeding 10 000 cubic feet, per quarter year, at the rate of Thirty Cents (30¢) per 1 000 cubic feet; for each thousand cubic feet of water in excess of 10 000 cubic feet and not in excess of 20 000 cubic feet, per quarter year, at the rate of Twenty-five Cents (25¢) per 1 000 cubic feet; for each thousand cubic feet of water in excess of 20 000 cubic feet and not in excess of 100 000 cubic feet, per quarter year, at the rate of Twenty Cents (20¢) per 1 000 cubic feet; for each thousand cubic feet in excess of 100 000 cubic feet and not in excess of 500 000 cubic feet per quarter, at the rate of Fifteen Cents (15¢) per one thousand cubic feet; for each thousand cubic feet in excess of 500 000 cubic feet per quarter at the rate of Ten Cents (10¢) per 1 000 cubic feet; provided, that the minimum charge shall be One Dollar ($1.00) per quarter year or part thereof.

(b) For any lot, parcel of land, building or premises situated outside of the corporate limits of the City of Dayton, Ohio, and having any connection with the City sanitary sewerage system or otherwise discharging sanitary sewage, industrial wastes, water, or other liquids, either directly or indirectly, into the City sanitary sewerage system, said charge shall be based upon the quantity of water used thereon or therein as the same is measured by a water meter there in use which meter shall be to the approval of the Director of Water, and there shall be charged for the first 3 000 cubic feet of water, or part thereof, so used per quarter year, the sum of One Dollar and Twenty-five Cents ($1.25); for each thousand cubic feet of water used in excess of 3 000 cubic feet and not exceeding 10 000 cubic feet, per quarter year, at the rate of Thirty-eight Cents (38¢) per thousand cubic feet; for each thousand cubic feet of water in excess of 10 000 cubic feet and not exceeding 20 000 cubic feet per quarter year at the rate of Thirty-one Cents (31¢) per thousand cubic feet; for each thousand cubic feet of water in excess of 20 000 cubic feet and not exceeding 100 000 cubic feet per quarter at the rate of Twenty-five Cents (25¢) per thousand cubic feet; for each thousand cubic feet in excess of 100 000 cubic feet and not in excess of 500 000 cubic feet per quarter, at the rate of Nineteen Cents (19¢) per thousand cubic feet; for each thousand cubic feet in excess of 500 000 cubic feet per quarter at the rate of Thirteen Cents (13¢) per thousand cubic feet; provided that the minimum charge shall be One Dollar and Twenty-five Cents ($1.25) per quarter year or part thereof.

(c) In the event a lot, parcel of land, building or premises discharging sanitary sewage, industrial waste, water or other liquids into the City sanitary sewerage system, either directly or indirectly, is not a user of water supplied by the Department of Water of said City and the water used thereon or therein is not measured by a city water meter, or by a meter acceptable to the Director of Water, then, in each such case, the amount of water so used shall be otherwise measured or determined by the Director of Water in order to determine the sewer service charge or rental provided in this ordinance, or the owner or other interested party at his expense may install and maintain a meter acceptable to the Director of Water for said purpose.

(d) In the case a lot, parcel of land, building or premises discharges industrial wastes, either directly or indirectly, into the City sanitary sewerage system, and the Director of Water finds that it is not practical to attempt to measure such wastes by meter, he shall measure such wastes in such manner and by such method as he may find practicable in the light of the conditions and attendant circumstances of the case, in order to determine the sewer service charge or rental, according to the corresponding rates per thousand feet provided in this ordinance.

(e) A discount of five (5) per cent. shall be allowed for prompt payment.

In addition to the charge or rental determined according to the rates specified herein further charge of five (5) per cent. of such amount so determined shall be added thereto in each case of failure to make prompt payment, and the total thus obtained shall be the sewerage service or rental in each such case.

The period within which payment shall be made in order to be classified as a "prompt payment" shall be determined by the Director of Water and shall be expressed through the rules or regulations which he is authorized by this ordinance to make for the purposes hereof:

> (f) The sewer service charge or rental provided in this ordinance shall be payable quarterly at the office of the Department of Water, and, at the option of the Director of Water may be made payable at the same time as water bills in the district in which the property is located are payable.
>
> (g) All lots, lands, buildings and premises located within the corporate limits of the City of Dayton, Ohio, and owned and used by the City School District for school purposes, all lots, lands, buildings and premises located within the corporate limits of the City of Dayton and owned and used by Montgomery County, by the State of Ohio and by the Federal Government for public purposes, together with all lots, lands, buildings and premises owned by the City of Dayton, are hereby exempted from said sewer service charge or rental, provided that nothing therein shall be construed as exempting the Dayton State Hospital for the Insane, The National Military Home for Disabled Soldiers of the United States, or The Wright Field, an aviation field maintained by the United States Government near said City from paying said sewer service, charge or rental.

Section 4. For any lot, land, building or premises from which connection is made with the City sanitary sewerage system or which begins to discharge sewage, any industrial waste, water or liquids into the City sanitary sewerage system, either directly or indirectly, after this ordinance becomes effective, a charge shall be made pursuant to this ordinance, the same to be a per diem pro rata amount based upon the minimum rate per quarter from the time such sewer connection is made, or such discharge into the City Sewerage system, either directly or indirectly, is begun, until the next following quarter period, except should the measured service exceed the minimum charge the corresponding measured rate or rates shall be charged.

Section 5. The charge or rental levied and assessed by this ordinance shall be payable in four quarterly payments in each year, as provided in Section 3, paragraph (f), hereof, provided that the amount of the rental payable for any lot or premises at the first payment period after this ordinance becomes effective shall be determined on the basis of a per diem pro-ratement of the minimum charge herein established beginning with the date this ordinance becomes effective, except that in those cases in which the metered or measured charge will exceed the minimum charge, the metered or measured charge shall be the basis.

Section 6. Each charge or rental levied by or pursuant to this ordinance is hereby made a lien upon the corresponding lot, land or premises served by a connection to the sanitary sewerage system of the city of Dayton, Ohio, and if the same is not paid within thirty (30) days after it shall be due and payable it shall be certified to the Auditor of Montgomery County, Ohio, who shall place the same on the tax duplicate of said county with the interest and penalties allowed by law and be collected as other taxes are collected.

Section 7. The charges or rentals levied pursuant to this ordinance shall be collected by the Department of Water, and the Director of said Department shall make and enforce such by-laws and regulations as may be deemed necessary for the safe, economical and efficient management and protection of the City Sewerage system and the sewerage pumping, treatment and disposal works, for the construction and use of house sewers and connections to the sewerage system, and for the regulation, collection, rebating and refunding such charges or rentals.

Section 8. The funds received from the collection of the charges or rentals authorized by this ordinance shall be deposited daily with the City Treasury and shall be accounted for and be known as the Sanitary Sewer Fund, and when appropriated by the City Commission, shall be available for the payment of the cost and expenses of the management, maintenance and repair of the City sanitary sewerage system and the sewage pumping, treatment and disposal works, and any surplus in such fund shall be used for the payment of the interest on bonds issued and outstanding or which may be issued to provide funds with which to pay the City's portion of the cost of constructing such sanitary sewerage system, or part thereof, and such sewage pumping, treatment and disposal works and to retire such bonds when they mature, and or for the enlargement or replacement of said sanitary sewerage system, pumping, treatment and disposal works, except that no part thereof shall be used for the extension of such sewerage system into unsewered areas.

SEWER RENTAL ORDINANCE NO. 23, OXFORD, OHIO[7]

An ordinance establishing the rates or rents to be paid to the Village of Oxford, Ohio, for the use of the sewerage system and the sewage treatment and disposal works by every person, firm, corporation or institution whose premises are concerned in connection with such system of sewerage and sewage treatment and disposal works.

BE IT ORDAINED by the Council of the Village of Oxford, Ohio:

Section 1. That there be and is hereby established the rents or charges to be paid the Village of Oxford, Ohio, for the use of the sewerage system and the sewage treatment and disposal works by every person, firm, corporation or institution whose premises are served by a connection to such sewerage system and sewage treatment and disposal works as follows: A rental equal to Twenty-five (25¢) Cents per 1 000 cubic feet of water consumed per quarter year, with a minimum charge of $1.25 per quarter year for each sewer service connection from each premise to the sewerage system.

---

[7] Estimated 1929 population about 2 200 for the village, and 2 500 for Miami University and Western College for Women. Passed December 2, 1924.

Said rents and charges are to become effective on the 2nd day of January, 1925, and the same are due and payable in April, July, October and January of each year, the first rental charges and rates being due in the month of April, 1925, and payable by the 15th day of the month wherein the same become due.

Section 2. Said rents or charges shall be collected and placed in a separate sewer fund and dispersed in the manner provided by law.

Section 3. This ordinance shall take effect and be in force from and after January, 1925, or the earliest period allowed by law.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1791

THE ENGINEER'S GROWING CIVIC RESPONSIBILITIES

ADDRESS AT THE ANNUAL CONVENTION AT TACOMA, WASHINGTON, JULY 8, 1931

BY FRANCIS LEE STUART,[1] PRESIDENT, AM. SOC. C. E.

There is no greater and no more lasting thing that we, as engineers and members of the American Society of Civil Engineers, can do to help the profession than to increase the esteem of the Engineer for his profession and the esteem in which that profession is held by the general public.

Our own minds and our own efforts are the forces that must bring these conditions about for us. Our own extra efforts can accomplish much, by applying our power to think to some of the essentials upon which our happiness depends, as well as to the technical matters upon which our living depends.

By the very nature of our work, the majority of our profession are salaried employees and are a part of some organization which we must take care not to disorganize. We must be deserving of the confidence reposed in us, and one of our problems is to work out a technique of service that increases our usefulness to our employers and yet makes us even more useful to mankind than we have been.

In centuries gone by when the various nations and parts of the world knew little of each other or what the earth's resources really were, all the energies of man and his helpmate were spent in providing the mere sustenance for life. As the people began to co-operate and to disseminate their knowledge, conditions began to improve until to-day there is enough for all, and will be enough for all in spite of any temporary depression.

By instinct, or by an urge not yet named, man began to lay up for the future, a part payment for what the past had given him. There are already paid for, by his efforts, and put away and recorded for the help of posterity, those things that are essential for coming generations to know that they may avoid many of the trials which beset his ancestors.

Engineers have made for themselves an economic position in our scheme of life. Their works are among the only physical examples we have of what has gone before; they have added their share to all the knowledge which we have recorded for coming generations; and they, to-day, are among the prime movers and the creators and constructors of the dreams and aspirations of our present civilization. This indicates that there has been something beyond mere living in their efforts.

---

[1] Cons. Engr., New York, N. Y.

I have stressed some of these things before, but I want to try to make clear some of the urge that is mixed up with your work for a living, the urge to help your fellow man, which is, in spite of yourself, inherent in your profession.

The Engineer more than any other class knows that to-day's civilization did not just happen—it represents the knowledge of the ages, brought along by recording progress as it was made, and passing it on. You and I do not have to start from zero on any proved subject, but the "how" and the "why" have been pegged and are our's for study. You owe a great debt to the past for what it has given you, and the only way you can pay it is by contributing your bit as you go along—giving of your time and talents to your State and to the World.

We live in an Engineering Age. Ancient and modern history and a cross-section of the civilization of the world to-day seem to justify such a designation of the period in which we live. As Engineers we have been and are engaged in one of the most important branches of the art of applying the sciences to improving living conditions, which have made such a valuation of world progress possible.

We have a right to feel that the profession is doing its technical part well, and if we can incorporate into our objectives a proper study of the needs of men as well as of materials, the power of our ideals will help us to take the place in world affairs which our extra efforts will deserve.

We owe to our profession and to our country a duty to be eager for public service and for greater usefulness, and we should with resolute co-operation take a serious part in the solution of the problems which beset us all.

There exists to-day a great need and an unparalleled opportunity for Engineers to use their powers of thinking and to make proper use of information and world trends, for the benefit of mankind.

The first requisite is that we, as individual members of our profession, Engineers young and old, must be more than able scientists, more than able inventors, more than able designers, more than able constructors. We must give the same quality of thought to matters of public progress and human interest that we give to technical matters and materials, and thus we shall make our technical judgment and advice more helpful in the counsel of men.

I have always felt that, while technical and creative work is a major phase of the Engineer's usefulness, it is not enough. In addition to contributing technical knowledge to help our prosperity, we should direct more of our individual energies toward taking our place with the forces which decide ways and means, and the uses to be made of our material progress.

This requires us to take a more commanding share in the deliberations or controls of the economic, financial, and managerial energies which influence our efforts or are influenced by our initiative, and to help them to plan a better scheme of world control of "supply and demand", and a better understanding of "cause and effect", than exists to-day. Thus, we may avoid a recurrence of such periods of depression as we now have.

We, as Engineers, should also take our part in the greatest problem of our age. This is to determine the direction in which living conditions shall be improved, and how the fruits of progress shall be stabilized and distributed in an equitable way to satisfy human wants, as they are brought about.

If we have these objectives to guide our ambitions, nothing is too great for us to attempt. If you will use only a part of the mentality with which you create material progress, to weigh and to formulate the policies upon which our happiness depends, you will become more useful to the world we live in, and you will yourselves enjoy the mental pleasures and the personal growth that come to the men whose aim is to improve the living conditions of the world.

Existing conditions make an urgent demand to-day for clear thinking by you, and men like you, in upbuilding and advancing our lagging prosperity. The reward for individual initiative and energetic effort has made our country great. This very incentive has been rewarded by such continued prosperity, that the herd instinct and avarice of men have run wild with our judgment, greatly aggravated world conditions, and brought on us all an unwarranted period of stress and strain.

The urgent question now for all is what should be done to relieve these depressing conditions in this country and abroad. The events of the past two years do not inspire us with confidence in the superior sagacity or ability of the forces that have been directing our prosperity, and we need to help ourselves.

Each one must draw his own conclusions as to how confidence can best be restored. In my opinion, the crash of prices came as a result of over-speculation, and the country suffered a loss of values and credit which had taken years to establish. The loss of such credit makes every one feel hard up and uncertain, and leaves men without the power to buy as freely as was formerly possible.

The prices of stocks, real estate, and materials are low because of a lack of confidence and credit which affects buying power. It is necessary that prices be maintained. Prices can be brought back to satisfactory values only by confidence that there will be a restoration of the business activities which have been swept away.

Too many are living on their capital, and these questions are economic, as well as political, and they seriously affect each one of us. If, in addition to the past decline of our credit, the twenty-five billions of railroad securities, including the bonds which are owned by the public, are too injuriously affected, it means a re-aligning of our whole scheme of American life.

Three matters are of great importance: Credit, which has not been controlled to prevent over-speculation; reciprocity, which should expand our markets; and a raise in railroad rates. Of these three, I believe that the raising of railroad rates would immediately change this country's situation for the better, until other remedies can become effective.

While it is out of the usual line of thought of the Engineer, I believe that a general lack of understanding and lack of publicity in financial affairs were largely to blame for present conditions. I see no reason why Engineers should not undertake a study of a part of our economic life that has failed us at a critical time. It seems unreasonable for us to put too much faith in the statement that the orgy of speculation could not have been stopped before it reached such a phase as to cause the depression which is just beginning to clear away.

The subject of the relation of general business to finance should receive our constructive thinking. Modern banking, at the moment, is based on the gold standard, and gold reserves are maintained at specified points, so that there can be no doubt that payment will be made. In different countries the ratio of the amount of gold and the amount of credit maintained is really a measure—among other things—of the lack of trust we have in each other's ability to pay.

Gold, except as a foundation of credit, is inconsequential as money tokens in the United States—it is arbitrarily about one-fourteenth of the money or credit we use in modern times. I am not convinced that the gold standard has had anything to do with the present troubles, but I think the misuse of credit has. I believe that credit is a power, and a force, having its flows and pressures, a force created by business and engineering and finance, reflecting the economic picture of our belief and confidence in collective efforts. Like every other man-made force, it should be under control, and while amenable to the pressures which come upon it from various directions, it should be made to regulate business affairs in other ways than by the mere lending of money, and to display the danger signals by withdrawal when over-speculation seems impending.

Credit has many angles and is admittedly difficult to measure and record. Its measurement should be a function of the Government in so far as it can properly require reports from institutions that handle other people's money. It should be taken out of the fog of mystery and multiplicity of detail and made understandable to the public. The making and destroying of credit could be better controlled by many minds than by a few, and there is no better way to control and extend its usefulness than by the sunlight of publicity.

If there is the real connection between credit and our material growth which I believe there is, and if important decisions must be made from interpretations of that relationship, then those predictions or interpretations should come from a source that all can trust as unbiased and without personal financial interest. Such a source would be the Treasury or Commerce Departments of the Government, and the interpretations should be published in as understandable a form as weather reports are, for all to read.

I am not unaware of the many charts of "this, and that", by formulas of "John Doe, *et al*", but they seem to be used for tips on the stock market or for price fixing, not for control. I want credit information collected by authoritative sources and used for objectives of value to the public.

If this theory is, or can be made, of service, why should there not be such credit reports in simple terms, and who shall say that such reports may not be of great use in the future in diversifying our efforts; in keeping our citizens employed; and in stabilizing proper distribution of our prosperity?

Aside from the regulation of credit, it seems to me one of the most important and helpful forces for permanent recovery would be the dissemination of knowledge, with a display of red lights at the dangerous points, or some method of putting it forward in such a forceful way that all must look, listen, and heed. This should cause a voluntary regulation of the surpluses of mass production, within the limits of credit available for these purposes. There must be a dissemination of knowledge which will cause a diversification of effort so as to meet the needs of the probable markets, and the citizens have a right to demand a political policy of reciprocity or commercial expansion which will broaden these markets.

As to world conditions, this country's international customers are kept out of the market by their struggles with the political diseases which beset the whole world. The net result here and abroad is a clogging of the equitable division and distribution of prosperity, which must be cleared if we are to maintain our standards of living.

Present conditions are distressing in these lines for the moment, but should improve quickly as the political diseases are cured and internal and world affairs are modernized, and the experience may be of lasting benefit to all if it changes the American's mental attitude as to his relation to world affairs and his permanent prosperity.

All human beings, of whatever race and nationality, are created with mental possibilities, and are susceptible of certain degrees of education and ambition. Yet two-thirds of the population of the globe are living on a scale that should be of grave concern to Twentieth Century civilization, and they form one of our unavoidable responsibilities.

In this country of ours, with an abundance of the necessities of life and a surplus of the products of industry, and the mentality to meet any world-wide competition in farm produce, industry, and transportation, there is no reason for an economic policy which permits five million men to be out of employment, while two-thirds of the world needs and will want our wares for generations to come for improving living conditions. Judging by past experience, they must, in fact, have them, or by stagnation be absorbed or perish.

In the early stages of the industrial life of our country, when education and immigration and an increase of population developed our chief consumers within our own borders, a protective tariff speeded the progress of the nation. The remedy now, I feel, is to call a halt on the present haphazard tariff policy and its financial fallacies, and cause a scientific application by engineers of a world-wide distribution of properly regulated surpluses, founded on a flexible scheme of world reciprocity, and a co-ordinated flow of credit to prevent abuses.

What is more, I believe reciprocity is as necessary a medium for the politically diseased abroad as it is for those at home, and that it would assist all in maintaining and improving living standards.

I feel that pioneering with reciprocity, by which I mean give and take in trade, and pioneering with education, by which I mean the spreading of the knowledge of the mental pleasures as well as of the physical pleasures our standards of living afford, in those sections which compose two-thirds of the world which is lagging behind the civilization we enjoy, would be of inestimable value to those backward sections of the world and would help us as well.

Before I discuss the subject of the railroads, let me give you my idea of what the railroads mean to us. I have served many of the best years of my life as a locating engineer and chief engineer and consulting engineer of railroads, and I feel that I with other Engineers have had a part in building the greatest monument made by the hand of man the world has ever known—the railroads of the United States and Canada. There is nothing in all human history which compares in conception and execution, immensity and comprehensiveness, and in useful and beneficial service to mankind, with our American railroads.

These railroads—from the discovery of how to use the steam engine to the present day; from the foundations of the structures of the roadbed to the roofs of the terminals; from the fenders of the piers on one ocean to the fenders of the piers on the other, and including the equipment on all their tracks—were made possible and executed by Engineers, so that what happens to them is of particular interest to you.

The railroad problem which confronts us to-day in our internal relations with our own people affects the most important single industry in our country and one we should keep in a healthy condition—an industry which to-day is being injured by favored competition and is badly in need of immediate assistance.

For the future, you might help by bringing your powers for engineering thought or research to bear on the subject of the railroads' relation to other competitive means of transportation, such as water service, and automotive service, the truck and bus, and private automobiles.

Water service (exclusive of that on lake and ocean) has its inherent limitations. It has but few points of production and distribution local to any particular canal or river facilities. For all other business not local to such facilities there is an extra cost of the transfer to and from railroads at the water terminals at each end, and such service cannot be of great moment in the railroad situation. The artificial stimulants, however, that will be used to create waterway activities are full of potential trouble and should be a matter of grave concern.

The automotive inventions have revolutionized transportation and brought on new problems.

The railroad has the advantage of car access to country-wide collecting and distributing points without the two transfer costs which are necessary in water-borne traffic en route to points not local to the water service. So, also, the truck, within limits of its economical use for main-road haul, and with depreciation and maintenance included, has the advantage of store-door delivery to innumerably more points of collection and delivery, without the transfer to trucks at both ends which is required by rail service to the same points.

Government or State subsidy of free roadbed for truck purposes can at best only lengthen the economical length of main-line haul. It cannot change the inherent flexibility of the service, which is the real competitive feature the railroads have to meet. With this in mind Engineers, for at least ten years, have been urging the railroads to use containers and make store-door deliveries, as a gradual transition to the time when they must operate the truck service themselves.

The bus situation is a different matter, and one of costs, and each route needs a careful comparative study of rates and facilities. Its disabilities, except for street traffic, are the discomfort and waste of mental energy of the passengers and the unavoidable risks of travel on highways, due to factors beyond the control of drivers.

Inasmuch as the public is entitled to, and should have, the cheapest and best means of transportation, these means, now competitive, must co-operate. Each particular agency, is entitled to its just place and use where it is found to be the most acceptably economic for all concerned. It must, however, be remembered that our railroads are the backbone of our daily traffic and must not be crippled by preferred treatment of other transportation facilities by water, by truck, or by other methods, which would be themselves unable to handle all our traffic or even any considerable part of it, and cannot take the place of the railroad service.

In my recent trips, I passed many trains with 4 000 or 5 000 tons behind one engine, and I could not help but feel that it was a dangerous policy to gradually whittle away the revenues of such an agency. We must stop it if we can.

A statement of the problem points to the answer. On account of its flexibility, the truck is here to stay, and the railroads will have to use the truck and bus business to protect their interests where profitable, as a part of their transportation service to the public.

As to the private automobile, it has added to our troubles psychologically, and it is a great modern problem with far-reaching effect on our lives. It has cut into the railroad revenue more than all other agencies combined; but it is such a great civilizer and business maker, as well as an unexampled agency for improving living conditions, that it is worth what it cost us. Means must be found to make good the loss of revenue to the railroads from this and other causes in some other way.

I have no doubt some means will be found to overcome the slump in railroad revenues, but until then our care should be that the usefulness of the railroads to the public has not been permanently injured before the remedies have been applied.

It seems to me that the quickest and most efficient aid to the railroads would be an increase in freight rate. The railroads are entitled to it, and inequalities could be corrected later. Then should follow an adjustment of taxes on a fairer basis, and a more liberal treatment of the railroads by the States and cities, in grade-crossing elimination matters. In due time the rate structure which has no simple rational basis, should be made the subject of scientific study by a technical body and should be taken out of the mystery class.

Railroad research is a fertile field; economies will be made in due course as they have in the past, but expenses unknown to-day will also accrue, and a net result is a time-consuming process. Under present conditions the main thing is that the railroads need help immediately, more for the general good of the public than for their owners or employees. With all their vicissitudes, they have been improving and cheapening their service to all from generation to generation. Nothing can take their place in the civic life of the nation. They are still the greatest asset for giving an impetus to prosperity. If we give them reasonable treatment the whole aspect of the situation may change in short order.

There are many other matters that might be discussed more fully, such as control of over-production which, when not controlled, caused surpluses which were never justified—a danger inherent in the use of the machine tool; or the need to diversify our efforts to meet the markets we have, which task was left to the other fellow merely because we were loath to change; or many of the mistakes that we as corporations or individuals have made; or even a discussion of the survival of the fittest. But the subject is too great for one discussion, and no full formulas will ever be found to prevent a partial recurrence of recent troubles.

Judging the present by the past, if we as a nation are willing to work, we have every right to tackle our difficulties with cheerfulness, but tackle them we must, and that is one of the reasons why I have discussed the matter at so much length. We must one and all help, each within his own capacity, for our standards of living are at stake.

These thoughts are my own and not a conclusion of the Society. In a case like this, individual thoughts are only suggestive in our scheme of things and are of value only if they can stimulate other minds to think. If we can find a unity of purpose, and change uncertainty into action, we can give some of the impulse of leadership that is needed. In any event I know we can help if we discuss these questions among ourselves. I can see that we, as a technical Society, have already helped to start trends of thought that will bear fruit. Gradually, we will be able to help evolve even more worth-while mental contributions and practical suggestions in planning to meet our economic needs.

There is still another duty to ourselves. From all sides, economic, social, and spiritual, present conditions warn us that it is important that we co-operate with all aidful agencies to make a proper technical, a proper laboratory, and a proper scientific, study of the greatest of all raw materials, man—man as an individual and man as a mass—and that we co-operate to counteract his weakness when found, and to make him, including ourselves, ambitious, energetic, helpful, and happy aids in the work of the world.

Please keep in mind for to-day and the many to-morrows which are to come, that the American Society of Civil Engineers is a great force and a great instrument for service. It requires our continuous working support and personal interest in its efforts in our behalf to keep it so. Therefore, let us all determine by co-operation, to co-ordinate our efforts and to keep the Society growing greater and greater.

By precept and example we should urge all Engineers to help each other by taking their places in technical societies, by becoming working members of their programs to advance and to help the profession, by recording their findings, and by doing their part in the many collective efforts of those societies.

In our collective relations, we should be historians as well as a part of the history of our age. The records of technical societies are our great collective contribution to coming generations. Such societies, together with universities and libraries in a broader sense, have been forming and developing and recording for posterity the educational thoughts and forces which have been advancing our civilization. These educational agencies have kept currently available our knowledge of engineering which has prepared the way, and which has enabled us and other Engineers like us to make those advances in the art of living, which will cause the present hundred years to be known in history as the Engineering Age.

The world at large, and even we ourselves, have just begun to realize the growing importance of our professional place in the scheme of life as it is to-day. I believe that the influence of engineering in the history of this age is so great that we can rightfully feel elation and encouragement as we move forward to meet our responsibilities of the future.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1792

"A NATIONAL RECLAMATION POLICY"

REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION[1]

WITH EXPLANATORY STATEMENTS BY MESSRS. J. B. LIPPINCOTT AND ELWOOD MEAD, AND DISCUSSION BY MESSRS. GEORGE W. FULLER, W. F. RUSSELL, NILS A. OLSEN, L. C. GRAY, FRANK ADAMS, M. J. DOWD, C. E. GRUNSKY, S. T. HARDING, H. H. DARE, CHARLES R. HEDKE, CHARLES KIRBY FOX, RHODES E. RULE, GEORGE M. BACON, G. E. P. SMITH, R. K. TIFFANY, DAVID WEEKS, JOHN A. WIDTSOE, F. H. NEWELL, A. D. LEWIS, JOSEPH JACOBS, F. N. CRONHOLM, R. I. MEEKER, E. B. DEBLER, JOHN E. FIELD, AND THE NATIONAL RECLAMATION POLICY COMMITTEE.

HISTORICAL STATEMENT

Under date of November 1, 1926, Joseph Jacobs, Chairman of the Executive Committee of the Irrigation Division of the Society, wrote as follows:

"The Executive Committee of the Irrigation Division at its last meeting authorized the appointment of a new committee to be designated 'Committee on A National Reclamation Policy'. In accordance with the thought that the Committee should be fairly large, should be well distributed geographically, and should be comprised of men of long experience in irrigation matters, its personnel was made up as follows:

"J. B. Lippincott, Consulting Hydraulic Engineer, Los Angeles, Calif., *Chairman*.
G. M. Bacon, State Engineer, Salt Lake City, Utah.
C. E. Grunsky, Consulting Engineer, San Francisco, Calif.
M. C. Hinderlider, State Engineer, Denver, Colo.
W. H. Code, Consulting Engineer, Los Angeles, Calif.
Burton Smith, General Manager, Twin Falls Canal Company, Twin Falls, Idaho.
G. E. P. Smith, University of Arizona, Tucson, Ariz.
J. C. Stevens, Consulting Engineer, Portland, Ore.
R. K. Tiffany, State Hydraulic Engineer, Olympia, Wash.
John E. Field, Consulting Engineer, Denver, Colo.
Joseph A. Elliott, Manager and Vice-President, Wyoming Development Company, Wheatland, Wyo."

---

[1] Presented at the meeting of the Irrigation Division, San Diego, Calif., October 4, 1928.

An effort was made to exchange views between these members by correspondence during the past year, but it proved unsatisfactory. A meeting of this Committee was called for the session of the Society in Denver, Colo., in July, 1927, but a quorum was not present, and no definite action was taken. The Committee, however, held a session in Los Angeles, Calif., on September 19 to 21, inclusive, 1927, at which were present: Messrs. G. M. Bacon, J. C. Stevens, John E. Field, W. H. Code, C. E. Grunsky, and J. B. Lippincott. As this constituted a quorum the Committee proceeded to business and adopted a "Progress Report" under date of September 21, 1927.

It was decided that supporting statements for this report should be written by the different members of the Committee to whom separate paragraphs were assigned and that an Editing Committee composed of Messrs. Grunsky, Code, and Lippincott should meet in San Francisco, Calif., about October 10 to review them.

Following the meeting of September, 1927, Mr. Code wrote a Minority Report which he submitted to all the members of the Committee for their consideration. In addition, a supporting statement was prepared by the Committee covering the Majority Report. The vote by letter of the eleven members of the Committee on the adoption of the report was as follows: Seven in favor of the adoption of the Majority Report, three in favor of the adoption of Mr. Code's substitute report. Seven members of the Committee favored considering the Majority Report of September, 1927, as a progress report which should be given further consideration by the Committee.

In accordance with the decision of the Committee to consider the report of September, 1927, as a progress report, a second meeting of the Committee was held in San Francisco on July 2 and 3, 1928. At that meeting there was present the following members: Messrs. C. E. Grunsky, W. H. Code, G. M. Bacon, G. E. P. Smith, J. C. Stevens, John E. Field, and J. B. Lippincott. This Committee reviewed both reports and the supporting statements and those present unanimously adopted a report dated July 3, 1928, which is a modification in some respects of the report of September, 1927. Copies of the modified report were sent to the members of the Committee not in attendance at this meeting.

REPORT

By resolution the Committee limited its work to a study of policies governing the reclamation of arid lands and related matters.

The policy of the United States and of the several States in the matter of water conservation and arid land reclamation should be controlled by certain basic principles, as follows:

(1) The waiving of interest payments to land owners on Government reclamation projects is unwise. In the future Government contributions should appear in the assumption of a part of the cost of project works and not in the granting of relief to the individual farmer such as the waiving of interest charges.

The United States Bureau of Reclamation has formulated a program of construction, covering the ensuing ten years, involving expenditures of approximately $100 000 000. To the extent that commitments have been made the Bureau should fulfill its assumed obligations and, on the other hand, the land owner should be required to meet his obligations or surrender his holding in the Government project.

(2) The conservation of the water in the rivers and lakes of the country should be under public control; and in order to lay a proper foundation for the making of comprehensive plans the Federal and State Governments should gather data, compile statistics, and conduct studies necessary to determine the feasibility of projects.

(3) The regulation of the flow of streams for the prevention of floods and for the best possible utilization of the waters should be undertaken by the States, or jointly by the United States and the States under such suitable forms of co-operation as may be appropriate under the Constitutional authority now delegated to each. They should prepare and adopt comprehensive plans for such regulation and should bear an equitable portion of the cost of water storage and flood-control work when the economic aspects after full investigations are found to be favorable, and the remainder of the cost should be allocated to flood control, irrigation, power development, municipal water supply, and other purposes.

(4) Where protection against flood waters results from the regulation of stream flow by means of reservoirs or otherwise, the proportion of the cost of the flood-control work not assumed by the Federal or State Government should be assessed against the lands and other properties which receive benefit therefrom.

(5) Municipalities or other public agencies or private parties should be allowed to construct approved projects in conformity with the approved plans; subject, however, to the public control of reservoirs and to the recapture after a reasonable time by the public of any franchise or similar rights conferred on private parties.

(6) The output of power and of water at Federal or State works should be disposed of at wholesale and not at retail.

(7) Interested States under suitable interstate compacts should be permitted by the United States to undertake the regulation of interstate streams.

(8) In the carrying out of further stream-regulation work, preference should be given to the construction of regulating reservoirs and the development of supplemental water supplies for existing irrigation systems, whether Federal or otherwise.

(9) Agricultural conditions due to over-production are such at present (1928) that it is undesirable for the Federal Government, except in the case of commitments already made, to bring new areas under cultivation.

(10) The construction of new irrigation projects should not be authorized except after thorough investigation and favorable recommendation by a Board of Review. This Board should include competent construction engineers, engineers with special operating and agricultural experience, economists, and

financiers familiar with local production and marketing conditions. The State should share in the responsibility for the selection and approval of projects. In determining the feasibility of proposed projects, State lines, local interests, and political expediency should not control.

(11) When new projects are authorized, principal and interest payments on construction costs should be required of the land owner. The interest rate should be low and the principal payments extended over a long period, with no payments on principal during the early years.

(12) The plan of repayment of construction costs of reclamation should be put into operation on each unit of the project at an early date after completion. The plan of payment should be sufficiently elastic to meet the settlers' ability to pay, but no relinquishment in the terms of repayment, once they are put into effect, should be permitted.

(13) In the case of reclamation projects, it should be recognized that settlement of the land is fully as imperative to success as construction. It can be greatly stimulated by the Government or other authorities taking drastic measures to prevent land speculation. The Department of the Interior is to be congratulated on its efforts to curb land speculation in recently authorized projects.

(14) Land settlement, including paternalistic, financial, or any other kind of assistance to the individual farmer, should be treated as a local matter and should therefore be made the concern of the State or locality rather than of the United States. Aid extended by Federal Land Banks has been generally helpful and the possibility of further extension of such aid by this or similar agencies is worthy of serious consideration.

(15) The Federal Government should continue its present policy of relinquishing control of completed works to suitably organized local agencies as soon as practicable.

(16) According to the report of the U. S. Bureau of Reclamation for 1926, the total area of land provided with water for irrigation was 1 803 000 acres in reclamation projects, of which, 1 320 000 acres were being cultivated by irrigation—a record of which the country may well be proud.

J. B. Lippincott, *Chairman*,
G. M. Bacon,
W. H. Code,
J. A. Elliott,
John E. Field,
C. E. Grunsky,
M. C. Hinderlider,
Burton Smith,
G. E. P. Smith,
J. C. Stevens,
R. K. Tiffany,
Committee.

July 3, 1928.

A NATIONAL RECLAMATION POLICY:
EXPLANATORY STATEMENT REGARDING REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION[2]

BY J. B. LIPPINCOTT,[3] M. AM. SOC. C. E.

In the preparation of its report on a National Reclamation Policy, the Committee of the Irrigation Division considered many points which it decided should not be elaborated or even mentioned in the report itself. The following explanatory statement is intended to give the report its proper setting. These remarks, therefore, may be considered as parenthetical or supplementary to the official report.

The Reclamation Act was passed on June 17, 1902, for the purpose of utilizing funds derived from the disposal of public lands, for the irrigation of arid lands in sixteen Western States. On June 12, 1906, the Act was extended to Texas. The law provides (Section 4), that,

> "* * * upon *the determination by the Secretary of the Interior* that any irrigation project is practicable, *he may* cause to be let contracts for the construction of the same. * * * The said charges [not exceeding 10 annual payments] shall be determined with a view of returning to the reclamation fund the estimated cost of construction of the project [without interest]."

The Act has been extensively amended. The provision for non-payment of interest has been retained, but the time of payments of construction charges was extended first to twenty, and then to forty, years. Extensive waivers of construction charges also have been made. In 1914, when Secretary Lane secured the Reclamation Extension Act, the law was changed and not only the adoption of new projects, but all appropriations therefor, had to be passed by Congress. Secretary Work on several occasions declined to construct projects so authorized, which in his judgment were not economically feasible.

In order to obtain a general perspective of the problem involved the following data are presented:[4]

Total area of crop land in the United States in 1924	391 647 372 acres
Crop land idle (8.7%)	34 007 716 "
On the Pacific Coast in 1924 the total area of crop land was	18 715 791 "
The crop land idle was 28% of this, or	5 201 519 "

---

[2] Presented at the meeting of the Irrigation Division, San Diego, Calif., October 4, 1928.

[3] Cons. Hydr. Engr., Los Angeles, Calif.

[4] Year Book, U. S. Dept. of Agriculture, 1925, p. 1380.

According to the report of the United States Reclamation Service for 1926 the total area provided with water under Federal Reclamation Projects was 1 803 000 acres, of which 483 000 acres, or 27%, was idle. It has been stated[5] as follows:

Irrigable land not now irrigated (in the United States)	30 000 000 acres
Wet land requiring drainage only	30 000 000 "
Wet land requiring drainage and clearing	60 000 000 "
Total	120 000 000 acres

From this it may be seen that the reclamation problem remaining is a large one.

The report of the Special Advisory (Fact Finding) Committee to the Secretary states[6] that up to June 30, 1923, the original construction cost of projects was $147 831 506. It further states: "The facts developed show that $18 861 146 will never be recovered. There will be a probable loss of an additional $8 830 000." The works built have been a credit to the Engineering Profession, the fund has been honestly administered, and the economic benefits to the West have been great. Such an extended experience is a justification for a review of the methods and results and for a discussion of the lessons that may be drawn therefrom.

Waiving of Interest Payments Unwise

As shown by the debates in Congress at the time of the adoption of the Act, and by the policies subsequently approved, Government irrigation enterprises were confined largely to projects which at that time were too expensive and required too long to construct to make them feasible as private enterprises. They then became feasible only by a system of investment that neglected the item of interest.

The extent of the subsidy due to this waiving of interest that practically has been granted to the water users on the Federal projects, as compared to the costs borne by settlers in State districts, or on private projects, is indicated by the following cost of United States Reclamation Work to June 30, 1922:[7]

Reported debits	$171 496 409
Reported credits	41 350 449
Reported net investment	$130 145 960
Corrected net investment (4% interest on annual net investments, compounded, to June 30, 1922)	70 706 685
Total cost	$200 852 645

---

[5] Year Book, U. S. Dept. of Agriculture, 1921, p. 430.

[6] Senate Doc. No. 92, 68th Cong., 1st Session, p. 53.

[7] *Bulletin No. 1257*, U. S. Dept. of Agriculture, p. 8, *et seq.*

The annual interest on the reported net investments plus interest on previously accumulated interest for 1922, according to the same authority, was about $7 725 000, or nearly twice the anticipated annual repayments on construction charges. Due consideration, however, should be given to the great indirect benefits that have been created by these works.

A computation of the amounts which would be due if interest at 6% were charged on deferred payments, shows that a farmer not so benefited would pay between 72 and 66% more for his irrigation works than one operating under the Federal Reclamation Law.[8] This does not provide for compound interest which would further increase the amount. Neither does it take into consideration the time during which water was supplied to farmers on Federal reclamation projects before payment of construction charges began, which in some cases has reached fourteen years.

According to the Fourteenth U. S. Census, 80% of the land irrigated in 1919 in arid parts of the United States was supplied with water by private enterprises that had received no public aid or endorsement; 6.5% had been developed by the use of Federal funds; 9.5% was served by State irrigation districts; and 2.7% was under Carey Act grants.

There is marked special legislation in selecting the settlers on 6.5% of the total irrigated area for the granting of such subsidies, while the settlers in the remaining 93.5% of the enterprises have paid full costs. If other irrigation enterprises were prosperous, such an aid to the settlers on Federal projects would not be so conspicuous, but it becomes so because many irrigation districts and private enterprises are struggling for existence.

It appears unfair that the farmers on 392 000 000 acres of land in the United States should be taxed for the purpose of supplying a large subsidy to those on only 1 803 000 acres, supplied with water by the Federal Government. This inequality is emphasized as there is an over-production in practically every branch of agriculture. It is the surplus, even though small, which breaks the price. The granting of subsidies to individuals also tends to destroy their ambition and to make them still further dependent. The report of the Special Advisory Committee (Fact Finding) in 1924, truly states (page XIII):

> "The Reclamation Service has retained the full management of all but two of the projects. This has not been satisfactory. The project management and the Washington office have become targets for criticism. A dependence on Federal paternalism has settled down upon nearly all the projects, and a corresponding bureaucratic tendency has grown up within the Reclamation Service. The water users have come to look upon themselves as wards of the Government, a specially favored class with special claims upon governmental bounty; and the Reclamation Service has been tempted to accept this definition of the water users. Nothing could be more detrimental to the progress of a venture which demands, first of all, individual courage and independence of the people concerned. The extension act provides that the operation and maintenance of the project may be turned over to the water users. This should be done at the earliest possible date."

The Bureau now recognizes that the management of projects should be promptly transferred to responsible local agencies; and this is being done.

---

[8] *Bulletin No. 1257,* U. S. Dept. of Agriculture.

Notwithstanding the waiving of this interest charge, large numbers of the settlers on the Federal reclamation projects have not paid their construction charges on projects costing on the average $57.75 per acre.[9] In addition, the "Omnibus Adjustment Act" intended to adjust water right charges, passed on May 25, 1926, made large deductions for construction charges on twenty Federal projects. Such conditions would not encourage the ordinary banker in making further loans to settlers as has been advocated in other parts of the Fact Finders report. Neither does it offer an attractive prospect for the repayment for projects now adopted which it is said will cost from $150 to $200 per acre.

It was the unanimous opinion of the Committee on a National Irrigation Policy that farmers on reclamation projects should pay interest charges on the cost of their works as do the water users on the remaining 93.5% of the irrigated areas of arid America. It is not fair to give the new settler preferential privileges above the water users on the State irrigation districts or under private enterprises. The rate of interest charged could properly be that at which money costs the Government itself, but the Government should not be called upon to sustain this loss in order to benefit individuals. The experiments tried in California and in the State of Washington do not indicate that paternalistic treatment of settlers has resulted in any particular benefit to them.

Under the California Irrigation District Act the bond issues which have been delinquent in the payment of interest and principal in 1927, when this subject was investigated by the writer, amounted to $4 627 000, on 95 810 acres. Their total bond issues amounted to $137 000 000, on 3 915 572 acres. The delinquency therefrom was but 3.4 per cent. The prevailing bond interest rate is 6 per cent. The California law provides the same general method for the sale of lands on which payments are delinquent as with other taxes.

The report of the Special Advisory (Fact Finding) Committee relative to Federal irrigation projects states (page XI):

> "On June 30, 1923, of the construction charges then due, 14.2%, or $2 537 222.46, remained unpaid, and of the operation and maintenance charges then due 17.6%, or $2 423 649.06, remained unpaid."

Of late, the Bureau has taken a stronger stand to enforce the payment of these charges.

In the case of California irrigation districts where the law is administered locally, payment of taxes is insisted upon and where there are no subsidies, better financial results have been obtained. However, in the arid area of the United States as a whole, 26% of all irrigation districts organized have failed in some period of their development, so that this method of procedure has also had its disasters. At first, these district laws were loosely drawn, and there was almost no supervision of their procedure in voting and selling bonds or letting contracts. In California, there are now close supervision and few failures. The Committee is of the unanimous opinion that a subsidy to settlers is not advisable. Where the Government has made commitments they should be honored, but the same fulfillment of contracts should be required of the water user.

---

[9] Senate Doc. No. 92, 68th Cong., 1st Session, p. 146.

The most flagrant examples of repudiation on Federal projects are said to be cases where local influences from the start undertook to secure public expenditures for reclamation with no intention of making repayments as required by their contracts. Some depended upon legal flaws, but others mainly relied on political influences. This movement made no progress in Congress as long as the Interior Department stood firm for the enforcement of the law and the contracts. These waivers are said to have been first covertly and then openly advocated in past years by the Department. When this succeeded, it of course broke the phalanx of resistance and many scrambled for such benefits. When the morale was broken the enforcement of contracts became more difficult.

It is a desirable and necessary public policy for the Federal Government to build hydraulic works whether for irrigation, drainage, or navigation, for the general benefit of the greatest possible number of citizens, rather than to make these allotments of funds for individual settlers on selected areas. This matter will be discussed further.

THE COLLECTION OF PHYSICAL DATA

Surveys of reservoir sites can be made promptly. Dam sites can be explored and distribution systems can be outlined quickly. However, the data necessary for a proper outline of a project, such as stream flow, rainfall, and temperature records, can only be obtained through years of continuous observation. Private enterprises are not adapted to the collection of such data; it is the province of the State and Federal Governments to make such long-time observations. The stream-gauging work of the U. S. Geological Survey is especially to be commended. It should be expanded. Some of the Western States are properly co-operating with the Federal Departments in collecting such physical data and in making essential topographic surveys.

REGULATION OF STREAMS SHOULD BE UNDER PUBLIC AUTHORITY

The determination by the Federal Government of the best comprehensive plan for the utilization of drainage systems where streams are interstate or navigable, is very important and such work is commended. It has adopted a broad policy of improving its navigable waters. Great expenditures have been made for this work. This is justified because of general public interest. Of late, many localities benefited have contributed to the cost of such improvements. The fact that certain individuals are especially benefited is not taken as the occasion for their direct assessment. A similar policy has been adopted for the improvement of main highways and large appropriations have been made therefor. Here, again, there is a co-operation between the Federal and local authorities. The Bingham Act provides for the aid of aviation in a manner similar to the Federal aid of navigation. As an example, the Federal Government now has a lighted air lane extending from New York, N. Y., to San Francisco and Los Angeles, Calif., and also one nearly completed from Los Angeles to Seattle, Wash. These air lanes are lighted at night and are supplied throughout their courses with emergency landing fields at Federal expense. The large cities served are requested to furnish the terminal airports.

Congress at its 1927-28 Session passed[10] "An Act for the Control of Floods on the Mississippi River and Its Tributaries and for Other Purposes". An appropriation of $325 000 000 was authorized. The Act provides that "no local contribution to the project herein adopted is required." This generosity, it is stated, is due to the extent of the problem involved, and because of large previous local expenditures. Section 4 provides that benefited localities are required to supply only certain minor rights of way. The aid of navigation is not the purpose of this Act, which contemplates the construction of regulating reservoirs. Section 3 provides for the States or levee districts to maintain the flood-control works after their completion and defines maintenance as including normally such matters as cutting grass, removal of weeds, local drainage, and minor repairs of main river levees.

These provisions are in distinct contrast to the proposed Act for building the Boulder Canyon Reservoir on the Colorado River where the full return of all funds to the Government is required by firm contracts in advance.[11] Section 13 of this Act also provides an appropriation of $17 600 000 for the control of the floods in the Sacramento River for which no return is to be made. This latter work is part of a co-operative act between the State of California, local land owners, and the Federal Government. Other great projects are proposed for the improvement of inland waterways at Federal expense and without return of funds.

The Federal policy of making large direct appropriations for public works for the general development of broad sections of the country, for navigation, flood control, highways, and aviation, has been established without repayment of the amounts furnished by it. Some such Federal policy should be adopted for the general benefit of all interests, including irrigation on navigable or interstate streams or their tributaries as well. Storage reservoirs and river-control works could properly be built in large part by Federal and State funds on such rivers as the Columbia, Sacramento, Colorado, Rio Grande, and their tributaries; and the floods could be regulated for all public use (be it irrigation, navigation, water supply, or power) and in such manner that the greatest possible numbers may be aided.

The Federal Government has already expended large sums for river-control works on the Lower Colorado by direct appropriations. Navigation in this instance is only a myth. If it becomes necessary to build an All-American Canal because of international complications to protect the irrigators of the Imperial Valley and to save just portions of the waters of that stream for the development of public and private lands in the United States, it appears fair that a direct appropriation should be made for that purpose. The people have done their full share in this effort, as well as in the projects of the Mississippi Valley. Established institutions are fully as deserving of help as new projects. This would be a broader and franker policy than the selection of 6.5% of the irrigated lands for special subsidies. The general plan for such works on interstate and navigable streams should be adopted jointly by the Federal and State Governments.

---

[10] Public Doc. 391, 70th Cong., 1st Session.

[11] H. R. 5773, 70th Cong., 1st Session, 4-b.

On non-navigable streams wholly within the State such works could properly be aided by the State itself, or by large conservation districts, or by the two combined. Some part of the cost of such works should be allocated to the interests benefited, but a substantial part of the expense for river regulation should be at public expense.

Construction of Works by Public or Private Agencies

Municipalities or other public agencies, or private parties, should be permitted and encouraged in building hydraulic works; but the control of large reservoirs built by Federal authority should remain permanently under its control because of the varied interests involved in their operation. This is the present provision of the Reclamation Act. When built as a part of a comprehensive plan, in so far as possible, they should be administered so as to produce the greatest general benefit. In the case of the Colorado River project the order of importance adopted is: First, river regulation; second, water supply; and third, power.[12] When reservoirs are built by private funds it is proper to provide for their ultimate recovery by the public as stipulated in the Federal Water Power Act.

Water and Power from Federal or State Reservoir Should Be Disposed of at Wholesale Rather than Retail

All the members of the Engineering Committee were agreed that State and Federal authority should not properly engage in the distribution of water or power to the individual consumers. The writer sees no reason why governmental authority should be restrained from installing the generating plants. It is not intended that water and power from Federal and State projects should be turned over only for distribution to private parties, but rather that it should be also delivered to either political subdivisions of the State or to mutual or private companies which have direct contact with the consumer. In cases, such as in the Salt River Valley Project, where an existing privately owned power distribution system was already in operation, it became naturally a medium for the distribution of power, just as the Salt River Valley Water Users Association with its network of canals became the natural distributor of water.

The Congressional Act of May 25, 1926, Section 46, provides:

> "No water shall be delivered upon the completion of any new project or new division of a project until a contract or contracts in form approved by the Secretary of the Interior shall have been made with an irrigation district or irrigation districts organized under State law providing for payment by the district or districts of the cost of constructing, operating, and maintaining the works during the time they are in control of the United States."

Section 45 of the same Act provides for a similar procedure in the case of amendments to any existing water-right contracts. While the water is to be delivered wholesale by the Reclamation Service to the district, its actual distribution to the consumers and the collection of bills are very properly to be undertaken by a local organization and not by the Federal Government. This policy is to be commended. Section 5 of the Act entitled, "H. R. 5773, 70th Congress, 1st Session", relative to the building of the Boulder Canyon Dam provides as follows:

---

[12] H. R. 5773, 70th Cong., 1st Session.

"That the Secretary of the Interior is hereby authorized, under such general regulations as he may prescribe, to contract for the storage of water in said reservoir as may be agreed upon, for irrigation and domestic uses, and generation of electrical energy and for the delivery thereof at such points on the river and on said canal and delivery at the switchboard to States, municipal corporations, political subdivisions, and private corporations, of electrical energy generated at said dam, upon charges that will provide revenue which, in addition to other revenue accruing under the reclamation law and under this Act, will in his judgment cover all expenses of operation and maintenance incurred by the United States on account of works constructed under this Act." (Section 4-*b*) "* * * and the repayment, within 50 years from the date of the completion of said works, of all amounts advanced to the fund * * * with interest thereon."

As far as the policy of wholesaling the water or power as provided in this bill is concerned, it is in accordance with the judgment of the Engineering Committee. However, the provisions of this Boulder Canyon Act are in marked contrast with the enormous appropriations for the Mississippi River where the obligation of the beneficiaries is to cut the grass on the levees.

Benefits from Public Funds Should Be Extended to All Water Users on the Stream

It has been stated[13] that 34 007 716 acres of the "crop land" of the United States were lying "idle or fallow". In the "Mountain States", which included Montana, Idaho, Wyoming, Colorado, New Mexico, Arizona, Utah, and Nevada, out of 25 604 416 acres of "crop land", 3 705 498 acres were "idle or fallow". The area of "land in farms" increased between 1920 and 1925, in the Mountain States, 12.4% and, in the Pacific States, it decreased 3.3 per cent. The average for the United States for this period showed a decrease of 3.2 per cent.[14] In 1920 there was a gain of 500 000 in farm population in the United States; in 1921, this gain was 200 000; in 1922, the loss was 460 000; in 1923, 461 000; and in 1924, it was 182 000.[15] The decrease in farm population in 1924 due to the city-ward movement was highest in the Mountain States, or 4.3%, followed next by the Pacific and West Coast Central States.[15] Between 1910 and 1920 the increase in the total population of the United States was 13 738 354, or 14.9 per cent. A table has also been published[16] giving acreages, production, and farm values from 1924 to 1925 which shows an estimated decrease in farm values of $570 663 000. From these data it appears that agriculture is unprofitable and farming consequently unpopular.

In view of these conditions, Secretary Jardine stated:[17]

"I am opposed to bringing new areas under cultivation until we have found a market for the products we are now producing. There are two things the

---

[13] Year Book, U. S. Dept. of Agriculture, 1925, p. 1380.
[14] *Loc. cit.*, p. 1379.
[15] *Loc. cit.*, p. 33.
[16] *Loc. cit.*, p. 1356.
[17] *Loc. cit.*, p. 22.

Government may do to prevent injudicious expansion of farm land. It may regulate its own land-settlement projects wisely and it may discourage undesirable private projects. There is a field here for useful and legitimate Government activity for the protection of agriculture and the promotion of the general welfare. I am as strongly in favor of such activity as I am against attempts to determine economic law by means of legislation."

In operating United States Reclamation projects, 29% or more of the land under canal is not farmed.[18] In California State irrigation districts, one-third of the land is uncultivated.

As water becomes more valuable, the methods of its use are improved. A substantial conservation of water in the future may be accomplished by the lining of canals. It is also being found that larger crops can be grown by the use of less water. Because of these conditions it is probable that extensions of existing canal systems will be made without increasing the volumes of water now being diverted. A notable example of such improved efficiency is in the Salt River Valley, in Arizona.

Due to the work of the U. S. Department of Agriculture there is an increase in the quantity production per acre in agricultural lands because of improved seeds. There has also been a great advance in efficiency of farm equipment which permits of deeper plowing and more effective harvesting. This progress is now well under way and will help, if it does not completely meet (from existing farm areas), the increasing demands for foodstuffs. This further relieves the immediate necessity for increasing reclaimed areas.

Throughout the United States, farm and orchard products have been low priced since 1923. While the prices on the staples, such as corn, wheat, and cotton, are determined by world-wide conditions, the crops that are best produced on higher priced lands, consisting largely of fruits and vegetables, are fixed in the home markets. A small excess of the supply above the demand breaks the price and conversely a slight deficiency stimulates it. During this period there has been an excess of the supply over the demand, especially with fruits and vegetables. This has resulted in losses and farm abandonment. This condition is one which should not be aggravated by the reclamation of new areas.

Nils A. Olsen, Assistant Chief of the Bureau of Agricultural Economics, U. S. Department of Agriculture, has stated[19] in part,

"A prosperous agriculture can not be achieved by over-development and over-expansion which injures not only established farmers but also new settlers. * * * The process of pushing agricultural expansion too rapidly tends to defeat its own end, if that end is the establishment of prosperous families on the land. * * * The idea of promoting residence in the country unless that residence can be maintained on a profitable basis to the family is to expect an impossibility and fly in the face of modern tendencies. * * * It is obvious that from many points of view it is much more economic to meet any demand for increased agricultural production by utilizing lands already in farms than by attempting to establish new farms."

It would be a much broader policy for the Federal and State Governments to aid the farmer who is now struggling with adversity on existing farms than to adopt a policy of reclaiming new areas whether by drainage or irrigation. There is a great need to reduce the difference between the price received by the producer and that paid by the consumer.

---

[18] Senate Doc. 92, 68th Cong., 1st Session, p. 26 (Fact Finders Report.)

[19] *Proceedings*, First Pan-Pacific Conference on Education, Rehabilitation, Reclamation and Recreation, Honolulu, Hawaii, April, 1927.

The argument in favor of the adoption of some of the major new projects which would result ultimately in increasing agricultural production has been stated[20] by Herbert Hoover, Hon. M. Am. Soc. C. E. (as Secretary of the Interior), in referring to the Lower Colorado River development in the hearings before the House Committee on Irrigation, as follows:

> "MR. HAYDEN.—The feeling in Congress is not very strongly in favor of reclaiming vast areas of land. It means in this instance a million acres.
>
> "SECRETARY HOOVER.—I think the answer to that would be that it would require approximately 10 years to complete the engineering work, and another 10 years before a complete use was made of the available water to those valleys, and, therefore, before production becomes very substantial there would be a period of 15 to 20 years. In that period the population of the United States will have increased twenty or thirty million, and that production will be very badly needed. I would be the last to wave the magic wand which would bring the production on at the present moment. It will probably take 20 years to complete."

It will be noted that Secretary Hoover agreed that present conditions do not justify immediate reclamation of additional areas, but he estimates that in 15 to 20 years their products will be required.

The policy adopted relative to the Mississippi River of protecting the farmers of the South who are now struggling with adversity from floods, is humane and proper. In a stream, such as the Sacramento, a dam could properly be built at Kennett, Calif., largely at Federal and State expense, which would sustain navigation, prevent floods, provide an adequate water supply for the existing farms in the Sacramento Valley and lower portions of the San Joaquin Valley, and permit of the generation of enough power to pay at wholesale rates for the greater part of the cost of the project. The distribution of both the water and power could be left to local agencies. Such a procedure would yield a State-wide and nation-wide benefit to as great an extent as the building of harbors or highways.

The Sacramento Valley is now staggering with debts of State drainage projects that probably cannot be paid. The limit of the Valley's irrigation water supply during the summer months has been reached, and navigation has been rendered impossible above Sacramento. The salt water from San Francisco Bay is encroaching on the reclaimed islands of the lower river. A comprehensive scheme involving the expenditure of large public funds for the betterment of such situations, in which the people of the entire West are involved, is commensurate with State and National effort.

Southern California now (1928) has a population of more than 2 000 000. It is increasing at a very rapid rate. Despite the fact that nearly all the surface flow of the streams in this region is used, about 80% of the drinking water has to be obtained from wells in Southern California proper. The groundwater supplies in most localities are falling. The only comprehensive source remaining is the Colorado River which is discharging in greater part in destructive floods into the sea about 16 000 000 acre-ft. per year, threatening the very existence of an irrigation district of 600 000 acres, followed by periods of inadequate low flow. By its control at the proposed Boulder Canyon Reservoir these adverse problems can be solved and made largely to pay for the cost of the river's improvement through the development of power, resulting in the great increase in National, as well as local, wealth. If this is not done there may be decadence instead of growth. The cost will be enormous; including the aqueduct, it may exceed $200 000 000. Surely such a project is worthy of both State and Federal aid. The financial plan proposed is fully justified, if not too liberal on the part of the local interests benefited.

---

[20] H. R. 6251 and 9826, 69th Cong., 1st Session, p. 49.

The cost of projects in the Northwest that have been adopted by Congress during the past few years, involving a construction outlay estimated at approximately $100 000 000 for the next ten years, are said to require an outlay of $160 per acre, or more. The water users thereunder are to be given forty years in which to refund this amount without interest. In view of the past history of the Reclamation Service, with projects costing less than one-half this amount, it is the prevailing belief that these repayments never will be made. The sum of $160 per acre is sufficient to acquire good farming lands in Illinois, Iowa, Kansas, and Nebraska, where schools, roads, and other public improvements already exist and where the markets for the products are at hand and large areas are idle. Under these conditions such projects should not now be built, particularly under the present condition of agriculture. Farmers are not asking for their construction. It appears to be a better policy to aid those who are endeavoring to develop areas now occupied and in need of help.

NEW PROJECTS SHOULD ONLY BE ADOPTED AFTER APPROVAL BY COMPETENT AUTHORITY

The existing law (Reclamation Act of 1902, Paragraph 4) states that the adoption of projects is dependent on the determination by the Secretary of the Interior that they are practicable. The Reclamation Service in the past, as well as at present, has in its organization many able engineers. The Secretaries have been competently advised by these men, whose judgment usually has been followed. Frequently, pressure has been brought upon Congressional representatives to force the adoption of projects. Congress, however, has now undertaken the adoption of projects. The Reclamation Extension Act (August 13, 1914, Section 16) now provides that expenditures for projects shall not be made except from annual appropriations of Congress. The Secretary of the Interior, in certain instances, because of adverse reports and his responsibility under the original act, has declined to construct projects authorized by Congress, which is a courageous and commendable attitude.

Obviously, no project should be adopted for construction unless it is physically and economically sound. The employment of competent advice both of an engineering and economic nature and its acceptance require no argument as to its propriety.

Interest Should Be Paid

In some Western States paternalistic measures under State supervision have been adopted for the establishment of settlers on the land, including the building of houses, barns, and the installation of machinery, the leveling and fencing of the lands, and the guaranty of power bills for pumping charges. The States have not been sufficiently prosperous to cause an expansion of this policy. While success has attended some, they do not appear to have secured any unusual prosperity. Where success has marked such enterprises, the conditions surrounding their initiation and subsequent progress appear favorable enough to have induced it without such paternalistic assistance.

The conclusion to be drawn from these experiences is that a liberal measure of paternalism will not insure success. Such aid will work injury in the long run to the social fabric that will far outweigh any temporary successes that may be credited to such a policy.

A National Reclamation Policy should be built around two fundamentals: First, the creation of equal opportunities; and, second, the stimulation of individual effort.

Any policy that supplies a prop on which the improvident may lean, and that fails to provide avenues whereby the energetic and thrifty can outdistance the less fortunately endowed, is foredoomed to failure. If such aid is to be given it should not be given by the Federal Government.

Repayments Should Begin Promptly

In the past, long delays have occurred after the deliveries of water before repayments for construction costs began. Based on the reports of the Reclamation Service, on four of the projects payments began as soon as water was delivered; on eight, the next year following. However, there are two projects in which in 1922, eleven years after delivery of water, payments had not begun and in one, the intervening period was fourteen years. A 10-year period with 6% compound interest represents 77% of the initial amount. At present, the Reclamation Service is insisting on prompt payments and is to be commended for so doing. Apparently, there is no good reason outside of political influence why these delays have been so protracted in the past.

The Fact Finding Commission properly recommends that as soon as water is provided for two-thirds of an area, the owners thereof should be obliged to begin making the payments. No long interval of time should elapse between the furnishing of water and the declaration by the Secretary that payments are due.

Any violation of this policy will deprive other settlers of equal opportunities and delay the operation of the revolving fund contemplated in the law. These delays put a premium on improvidence and are unfair to those who are able and willing to meet their obligations. The terms of repayment should be adjusted to the reasonable ability to pay. The Department of the Interior has adopted[21] the policy of decentralization and has executed contracts with thirty irrigation districts and water users' associations on sixteen projects for the transfer to local control of the care and operation of the irrigation works of these projects. This, it is stated, has resulted in a better morale and financial standing on the projects.

---

[21] Annual Rept., 1927, p. 20.

LAND SETTLEMENT ESSENTIAL TO SUCCESS

On most new irrigation projects, whether Federal, State, or private, colonization is a serious problem. It is difficult to induce people to settle these new farm lands because generally, under present conditions, agriculture does not pay. If farming were profitable the colonization problem would largely disappear. This same argument applies to the difficulty in financing the farmer. Farming will not pay until the demand for foodstuffs catches up with the supply.

The bane of many irrigation projects, whether publicly or privately conducted, has been land speculation. If the dirt farmer can be given access to the land at its base price before inflation, his chances of success are better than if he has to pay a speculative price for it.

On the Pacific Coast, real estate agencies, with "high pressure" salesmen, frequently charge from 25 to 35% of the sales price of the lands for their commission, and require that their fee shall be paid from the first instalment of the purchaser, the real land owner usually having to wait for his compensation from deferred payments. This is hard on both original land owner and the settler. It is very difficult to overcome such speculations.

The policy now being enforced on new projects may offer a greater stimulus to the proper settlement, than any colonization plan previously attempted. Drastic measures are taken to discourage land speculation prior to the adoption of the project, by fixing sales prices of lands therein. A vigorous policy in this regard will produce a wholesome effect. The Government or State can properly insist as one of the conditions of its undertaking a project that every surplus parcel of land within the proposed district be listed with an authorized real estate agency, at a stipulated price that contains no unearned increment. The Government or State, in these recent instances, properly reserves the right to fix the price of such land. These tracts are to be offered to prospective settlers at such prices plus sufficient commission to cover overhead expenses only.

Every sale contract should be between vendor and purchaser, the Government merely acting as agent. In case of re-sale, the land must be offered through the same agency for a certain period of years until the project is safely on its way to prosperous settlement.

The Department of the Interior is to be congratulated on its efforts to curb speculation on its recently authorized projects.

PATERNALISTIC AID

It has been recommended that the water users on Federal projects should be further aided by advancing loans, leveling lands, etc. It was the judgment of the Committee that if such paternalistic system is to be extended it should be through local organizations, such as possibly the State or county where there would be a more direct interest in the return of funds and where neighbors would pass on the ability and intent of the borrower to make returns following lines perhaps as adopted by the Farm Loan Banks.

THE ACCOMPLISHMENTS OF THE RECLAMATION SERVICE

The Reclamation Service has constructed 120 dams, built 16 000 miles of canals, laid 3 700 000 ft. of pipe line, and has excavated 252 000 000 yd. of materials in the performance of its duties. In 1925, the estimated crop values produced on these projects was $77 608 880 on a total area of 1 432 155 acres. The engineers who have been connected with this work may justly regard it with pride. The report that the Committee has presented is not a criticism of this service either past or present. The policies have been fixed by acts of Congress and not by the engineers and executives. This report is intended to be a review of the operation of these policies and an expression on the part of those who are interested and somewhat acquainted with the results.

A NATIONAL RECLAMATION POLICY:

ECONOMIC ASPECTS OF FEDERAL RECLAMATION[22]

BY ELWOOD MEAD,[23] M. AM. SOC. C. E.

The report on a National Reclamation Policy prepared by a Committee of the Irrigation Division deals with a problem on which Government officials charged with the Reclamation work may quite properly hold other and contrary views to those expressed. This paper comments on the report as from this angle and, therefore, may be considered as voicing the essentials of another side of the question.

A COMPLEX UNDERTAKING

The reclamation of arid land by irrigation is not a single or a simple undertaking. It involves the construction of works, which is engineering. It requires the settlement and cultivation of land, which is economics. It is not a success unless it creates communities of happy, prosperous homes, which is a social problem. The report under consideration was prepared by engineers. If it had been prepared by economists more attention would have been given to settlement and the creation of conditions which would enable the money spent on construction to be repaid. If it had been written by settlers more would have been said about their needs. Investigations and discussions like those inaugurated by the Society, therefore, will promote an understanding of reclamation and make future development even more valuable.

The influence of Federal reclamation on agricultural development has varied widely in different States. It has been least in California and greatest in Idaho and Arizona. The rich and populous irrigation districts of these two latter States are the creation of Federal works. Outside of California important irrigation works of the future will be built by the Government. Costs are too great and needed income will be too long delayed to make such development attractive to private enterprise. Government projects must continue to be subsidized either by not requiring interest on construction costs, as at present, or by the Government paying a part of the cost, as is proposed in the report.

The present (1928) income for building Federal irrigation works comes from four sources: (1) Payments by water users; (2) payments for power; (3) income from sales of public lands; and (4) a percentage of the income from oil leases. The first two are increasing; the last two are diminishing. The total is about $10 000 000 per year.

The works being built are large and costly. Their completion will require several years. Farm development must await the water supply; hence, there is small prospect of any material increase in irrigated acreage during the next decade. No contracts for new construction can be made until approved by both the Secretary of the Interior and the President. This operates as a further check on rapid development.

---

[22] Presented at the meeting of the Irrigation Division, San Diego, Calif., October 4, 1928.

[23] Commr. of Reclamation, U. S. Dept. of the Interior, Washington, D. C.

The Second Stage in Reclamation

There is a gap in reclamation between the completion of canals and the use of water in irrigation. The first step in reclamation is to provide water. The second is to bring it into use. This requires settlers for the uncleared, unleveled land. It requires preparing that land for irrigated culture, erecting farm buildings, and growing crops on soil baked for centuries. The cost, the hardships, and the anxiety of this second step, were always greater than was realized or admitted, and the cost is now two or three times what it was fifteen years ago—say, in 1913. An irrigation canal with unpeopled farms below it is a liability, not an asset. Income and the benefits of reclamation are realized only when the second stage of reclamation is completed.

Other obstacles to carrying out settlement and farm development have become more serious in recent years. The pioneering spirit which led settlers to do the difficult and unremunerative work of clearing and leveling the land is gone. The open country no longer appeals as it once did. The opportunities of other industries are much broader. The cost of changing raw land into farms is now so heavy that money or credit is usually needed to supplement the settler's meager capital. Economic surveys of developing reclamation projects made by committees which included practical irrigators, economic experts from agricultural colleges, and representatives of the Bureau of Reclamation show that from $5 000 to $10 000 must be spent to provide the permanent improvements and equipment of an 80-acre farm.

The percentage of home seekers with capital enough to improve and equip their farms without borrowing is very small and the number willing to invest their capital in a development of this character is still smaller. They can get more for their money by buying improved farms in established districts. The settlement and development of unimproved, unpeopled areas depends largely on tenant farmers, to whom the lure of ownership is strong, and to the sons of farmers. Such applicants rarely have more than $2 000 to $3 000, and settlers with less than $5 000 capital will need to borrow money to make their farms produce a living income. There are few projects where this money can be borrowed on terms which the farmer can meet, if it can be borrowed at all. On nearly all developing projects loans are for short periods, with interest rates which are higher than agriculture can stand.

On four projects, which have been in operation for more than ten years, those who have improved farms are prospering; but more than one-half the land is unirrigated, and 800 more settlers are needed to bring all the land in these projects under cultivation. If one-half this number of settlers could be secured it would insure the economic solvency of the Government works and the payment of the Government's debt. These settlers could be secured if there were some agency to loan money needed to erect inexpensive farm buildings and prepare the land for irrigation. They cannot be secured without some credit aid not now provided. Short-time loans to buy live stock can be secured, but money for permanent development is not available.

This credit phase of settlement has been emphasized because it is a stumbling-block in the way of success. It is also a menace to the solvency of works now building or to be built. On one project, where the irrigation works will cost $11 000 000, there is no anxiety about the payment of construction costs on more than one-half the area. The farms are improved, the land has been prepared for irrigation. The owners have contracted to pay full construction costs on their entire area; cultivation and use of the water are assured. On the unimproved, unpeopled part of this project, however, the surface of the land is uneven and covered with brush or with second-growth timber. To prepare this land for irrigation, and to provide the necessary buildings and equipment, will cost from $75 to $150 per acre. If this money could be advanced and payment could be spread over 20 years, with 5 or 6% interest, buyers could be secured; but they would be reluctant to take on a clearing and leveling job for which they would have to provide all the money.

This project presents new economic problems, the solution of which needs new policies and new laws. Some of the land belongs to the State, some to a railroad company, some to individuals, and some to the United States. If these owners act together and the preparation of the land for cultivation is put into competent hands, and farm boundaries are fixed to agree with the topography of the country, the land can be made ready for cultivation in less time, at less cost, and with infinitely better results than can be accomplished by leaving this to the action of the separate owners. Each will wait on the others, development will be delayed, and money will be lost. The heaviest loser, of course, would be the Federal Government.

On another project the irrigation works being built have an estimated cost of $18 000 000. A considerable part of the land under the project is now being irrigated from pumps. The cost of pumping has so increased in recent years that it is now greater than irrigators can afford to pay. A gravity supply will be much cheaper and therefore will avert failure of settlers and bankrupt communities. There is no anxiety about water payments where the lands are settled and improved. Payments will begin, and the water will be used as soon as it is available; but there is a financial problem on 70 000 acres of unleveled, uncleared, and unoccupied land which forms a part of the scheme. The owners of this land are widely scattered. Hardly any of them expect to become irrigators. They wish to sell. There is little danger of inflated prices. The land has been appraised and the owners have agreed to sell at the appraised price, which is nominal. Cheap land, however, does not insure settlement and cultivation. Here, as in the other case referred to, credit and co-ordinated action are necessary. Part of the land is owned by the State, part by purchasers of State land grants, and part by men who acquired it under the homestead, grazing, and similar acts. A small fraction is owned by the United States. If it is necessary to wait for these independent agencies to improve and develop these farms or find settlers to do it, it will be many years before the water made available is used and before construction costs are returned to the Reclamation Fund.

Anxiety in regard to the financial outlook is met by the statement that the Government is protected in the repayment of its expenditure by district contract. It is true that all the lands in these projects are obligated to pay the entire construction cost and this obligation is a first lien on all the land, but it would impose an undue burden on the developed lands to require them to pay the charges against the undeveloped lands. A part of the project could not carry the cost of the whole. As a practical matter, solvency and settlement are closely related.

The measures for aiding settlers in farm development, suggested in the Committee's report, are entirely inadequate. State aid has been sought. Legislation to require this has been considered in Congressional committees, but investigation showed that some States lack the means to extend this aid, some are prevented from doing so by Constitutional prohibitions, and in every State there is small prospect of political support for this co-operation. The Federal Land Bank has not been, nor is it likely to be, of any assistance in changing raw land into farms. It does not loan money except on income-producing farms, and that means loans are withheld until after the problems of settlement and farm development have been solved.

There is great reluctance in Congress to do more than the Government now does. The reasons for this reluctance are not to be ignored, and it is desirable that further action be avoided and that settlers be found with money enough to make their own improvements and buy their own equipment. There appears to be little hope of this on some of the older projects or on some of those now building. It is the writer's conviction that one of two things should be done: (1) Either provide aid in carrying out the second stage of reclamation; or (2) quit building canals to irrigate unimproved land.

The latest economic report on settlement and farm development recommends that the Government purchase all privately owned, unimproved land on a project before construction begins. If this were done, it would forestall land speculation, enable roads and ditches to be laid out to meet the needs of irrigation, and prevent the imposition of heavy State and county taxes before development is completed. Whether this action is taken, it is desirable that the States be more co-operative than in the past. The rule now is to raise taxes as soon as construction begins. This imposes a burden on the owners of partly improved farms that makes payment of construction costs difficult and sometimes impossible. The heavy tax burden is one of the arguments used for political action in writing off or postponing payment of construction costs.

A bill now (1928) before Congress is intended to provide a laboratory test of aid and direction in settlement. It authorizes the appropriation of $500 000 from the Reclamation Fund. It limits the expenditure on any single farm to $3 000 and restricts loans to provide for the erection of farm buildings and the preparation of land for irrigation, the money thus advanced to be secured by a first mortgage on the land and to be repaid with interest in installments extending over twenty years. If anything is done it should be done with care. If the $500 000 is authorized, not more than $100 000 should be appropriated in any one year, and no further authorization should be made until at least one-half these advances has been returned to the Reclamation Fund. This might be arranged through loans to settlers from the Federal Land Bank. Improvements made from these advances would make Federal Land Bank loans possible.

There is also a fear that lending money to settlers would lessen their initiative and self-reliance and tend to make them leaners on the Government. That objection was made to the Federal Land Bank legislation. The same objection applies with greater force to building canals by the Government. It might be said, "Let settlers build their own canals and reservoirs." The answer is that this is impossible because of their cost. The high cost of changing raw land into farms makes it impossible also for worthy, industrious home seekers to get started without aid. There is a kind of aid which does undermine independence and self-reliance, and another kind which strengthens hope and inspires effort. That is what wisely directed credit in farm development would do.

ACHIEVEMENTS OF FEDERAL RECLAMATION IN COMPLETING PRIVATE DEVELOPMENT

It is well to emphasize the gratifying social and economic results which have followed the Government's activities in supplementing or completing district or private development. By taking over the canals of Salt River Valley and building the Roosevelt Reservoir, Federal reclamation rescued discouraged and helpless irrigators and made Salt River Valley, Arizona, one of the most prosperous irrigated sections of the country and a great economic asset of the State. Without similar action the Carlsbad Project in New Mexico would now be only a memory.

In the Weber Valley, in Utah, a reservoir is being built (1928) to furnish water for late irrigation to intensively cultivated land. It will more than double the value of the crops grown. This additional water supply may be likened in value to the water which puts out a fire. It will save disastrous losses in dry seasons and make possible greater profits in all seasons. There is no worry about the payment of construction costs. The water will be used and the payments made according to contract.

The Government has built two reservoirs on Snake River, one in Wyoming and one in Idaho. It is difficult to state adequately the contribution this has made to the success of irrigated farming in Idaho. Only the Federal Government could have co-ordinated the different interests necessary to carry out a scheme extending over State boundaries. A part of this stored water is being used to give an additional supply to 80 000 acres of land first developed and settled as a State irrigation district enterprise. After farms had been improved and prosperous towns had grown up, it was found that the water supply was inadequate, and ruinous losses from failure of crops were suffered in dry seasons. Towns were shrinking in population and business and farms were being abandoned. The Government, by providing what was impossible to local effort, has restored confidence and prosperity. In the Snake River development there is no uncertainty about the repayment of Government cost. A part was repaid in advance by the water users.

The improvement of an old development and the complete utilization of the resources in land and water through an entire drainage area, are being carried out in the Yakima Valley, Washington. This kind of development is specially suited to the Federal Government. It is solvent and beneficent and there is a broad field for its extension.

The passage of the Adjustment Act in 1926, inaugurated a new era in Federal reclamation. The necessity for this legislation grew out of the hard times which followed the World War. It made it impossible for farmers on some projects to meet their payments. When owners of improved farms in Iowa and Illinois were being sold out, farmers on partly improved farms, under reclamation canals, found their expenses greater than their incomes. It was impossible for them to recover under existing contracts. Congress recognized this situation and passed an Act under which the annual payments of indebtedness in some districts were extended from 20 to 30 and 40 years. The land in all projects was classified in accordance with scientific soil surveys; payments on unproductive lands were cancelled; land injured by seepage, or which from other causes would not at the time grow profitable crops, was given temporary relief from payments. The loss to the Government from this action has been greatly exaggerated. The permanent loss was estimated at $14 667 965. Some of this will be recovered. The temporary loss was fixed at $12 788 406. There are hopeful indications that a large part of this will be paid.

Delinquent payments were added to the construction debt. The Reclamation Bureau was authorized to employ economic experts and practical advisers. The foolish idea that practically every one can succeed as an irrigation farmer was discarded and provision was made for examining home seekers by a local board. This has proved a protection to the inexperienced and over-sanguine and is giving reclamation a fair chance to show its value. The Secretary of the Interior has required settlers to have not less than $2 000 in money or equipment and this leads them to investigate how much developing a farm will cost and to plan their operations with more care.

This Act authorizes the transfer of projects or parts of projects to the water users. Eighteen districts under ten projects have been turned over to local control in the period, 1926-28. Where this is done the Government is relieved from any further expense for operation and maintenance. These changes have promoted co-operation and good feeling between water users and the Government. They have improved the morale of projects and helped to increase construction payments. In 1927 these payments were more than $1 000 000 greater than in any previous year. Delinquencies once so alarming have almost disappeared. Farm practices are improving. More valuable crops are being grown. There are more acres of sugar-beets, more dairy herds, more farm flocks of sheep, more poultry, and more market gardens. In 1927, the crops produced on the 2 504 046 acres irrigated from Federal works were worth $133 207 210, which is an average of $53 per acre, or two and one-half times the average acre value of crops in the United States. The value of this one crop almost equals the entire debt of settlers to the Government. Two such crops will be worth more than the Government has spent on reclamation since the Act was passed. Another crop of equal value is expected this year (1928).

To complete existing projects in accordance with the 10-year program submitted by the Secretary of the Interior to the President and Congress in 1926 will require about $100 000 000. As the average income of the Reclamation Fund is about $10 000 000 per year, there need be no nervousness about Federal reclamation increasing the agricultural surplus during the next decade. The endorsement in the Committee's report of this conservative plan of development and of the other economic changes wrought by the Adjustment Act is greatly appreciated.

In its limited field reclamation is one of the most difficult activities of the Government. The Act requires that all money spent shall be repaid. That is impossible, although the Reclamation Bureau has striven to live up to it. Each new project presents new conditions and requires the overcoming of new obstacles. Dams have to be built in remote localities. The suitability of crops to soils has to be tested. Communities have to be organized and markets established. Reclamation has led to a creation of wealth in land many times the cost of the works. Its contribution to other industries, to commerce and trade, entitles it to a credit and support it has not always received. From one town on a Federal project where twenty-five years ago there was nothing but range cattle, a single railroad in 1927 did a business of $800 000. The indirect benefits from reclamation include help given in solving the problems of soils and climate, improving irrigation practices, founding rural communities which are an economic and social strength to the nation, and creating a wealth in land many times the outlay for works.

DISCUSSION

George W. Fuller,[24] M. Am. Soc. C. E.—This report is of great interest to all who have occasion to consider present or prospective reclamation projects. Obviously, there is more irrigated land than is now needed to supply farm products to the population of Continental United States. Unless a tariff schedule can be adopted with other arrangements such as to promote the successful sale of farm products abroad, the future of irrigated lands in the Continental United States will depend largely upon the relation between supply and demand as affected by increases in population. Therefore, care should be exercised to guard against the development of irrigation projects tending to cause a continuance of excess production of farm products as compared with market demands.

It is doubtful whether American citizens in general realize how this question of excess production is being affected by a marked decrease in the rate of population growth in the Continental United States. Statistics bearing on this subject have been analyzed by many writers and are well summarized by Dublin.[25] Since the first Federal census in 1790, the United States has doubled in population on an average once in 27 years. During the middle of the Nineteenth Century, however, when immigration from Central and Western Europe was abundant and the growth of the colored population in the Southern States was very rapid, it took only 20 years for the population to double.

The situation is now changed. Excess of birth rate over death rate according to the 1920 Census records showed an increase in population of about 11 per 1 000, or 1.1% per year. This indicates that it would take about 60 years for this country to double in population; that is, on the basis of uncorrected birth and death rates, which vital statisticians say should be adjusted so that the influence of existing conditions will have its due *pro rata* effect on birth and death rates for groups of persons of different ages.

The slowing up of the increase in population in this country is related to many factors. Among them may be mentioned the restriction, since the World War, in European immigration and changed viewpoints in the home with its complications as to servant questions and the continuing custom of the native stock to have small families. One of the effects of pre-war unrestricted European immigration followed by present restricted immigration is that there exists in the United States an unusually large proportion of female population of child-bearing age. Continuance of restricted immigration will prevent increments to this group from such source and hence the group as a whole will become relatively smaller with a lessening birth rate in the future as compared with recent years when the effect of pre-war immigration is still felt.

Without going into great detail it is sufficient to state that when consideration is given to the foregoing item; to the statistics of life expectancy of persons in each group, arranged in ages; and to the effect of present environment with its modern sanitation, preventive medicine, and personal hygiene, reconstructed birth rates and death rates are derived which indicate that it will take this country about 120 years to double in population. In fact, there are prominent statisticians who state that the population of this country will become stationary with a maximum about 200 000 000 people by 2100. Others on the contrary predict, on the basis of ultimate possible food production, a maximum of 300 000 000. Whatever may be the precise outcome as to population growths it is certain that there is nothing in sight to indicate that an increased demand for farm products justifies the early development of large new irrigation projects.

---

[24] Cons. Engr. (Fuller & McClintock), New York, N. Y.

[25] "Health and Wealth," by Louis I. Dublin, Harper Bros., 1928.

W. F. RUSSELL,[26] ESQ. (by letter).—The engineering problem of reclamation has been well taken care of to the extent that great irrigation systems have been completed; but the building of a system of canals is only the beginning of reclamation. To say that the responsibility of the Government ends with the building of canals may be compared to the manufacturer of an automobile contending that his car was finished without supplying it with wheels. The preparation of the land to receive the water and the placing of settlers to till the soil is the most important part of the reclamation of arid lands. The money invested in dams, reservoirs, and canals is wasted if this other part of the work is not done just as scientifically and well as was the planning and construction of the canals. In most cases the cost per acre for land preparation and settlement will exceed the cost of construction.

The advancement in the scientific pursuits has been very rapid, and irrigation reclamation is no exception. The man who is to succeed on the land, who is to utilize the water that comes through the canals, and who must produce from the land through the proper use of the water, must be a scientist in every sense of the word. He must acquire more scientific knowledge to be a good farmer than to conduct any other business, and it is time that the people of the United States realized not only that farming is the most important of human activities, but that the farmer is entitled to, and should receive, just and fair compensation for his investment, for his labor, and for his responsibility. Too little importance has been given to the farmer by the leading men, both in Government and financial circles.

If the projects that have been built by the Government are to succeed, the Government must carry the responsibility of completing the work, and that cannot be effected until very acre of land under the canal is producing. The settler cannot finance the preparation of the land. It requires a number of years as well as capital expenditure, to develop raw land to its maximum capacity of production. It also requires considerable capital for a farmer to support his family through these years of development.

The writer has found that a settler going on to raw land with $2 000 to $3 000 is financially embarrassed before the land can be developed to a point where it will produce a comfortable living for his family and begin to pay the capital charges against the land. This condition must be overcome by some policy of finance. It may be a loan against the land, or it may be placed as capital cost and be added to the price of the land. It does not matter how it is done. It is the responsibility of the Government to protect the huge investment it now has, and to protect the interest of the settler, that will make the project a success.

---

[26] Field Superv., Colonization Dept., Lethbridge Northern Irrig. Dist., Lethbridge, Alberta, Canada.

The Government policy of financing construction is to be commended, but it did not go far enough. There are those who might say that this would be charity and that, if the Government were to undertake to supervise the development, it would be paternalism. Yet they are willing that supervisory methods be used for the construction of the dams and reservoirs, and the laying out of the canals; furthermore, they are willing that the same methods should be used in the construction of the power plants and lines of distribution; and that other lines of industry should use the same methods. They will agree that it is necessary that a policy of financing must be instituted in order that these industries should succeed; but if the same principles are applied by the Government which has already gone a long way with the work of reclamation after all other agencies would go no farther, it is charity and paternalism.

The problem of proper development of the land already under Government canals and the settlement of the same by a class of people fitted for the work is one that will require more skill and closer supervision than any part of reclamation the Government has ever attempted; and until some policy for successfully financing the unoccupied land is worked out and put in operation, the great reclamation problem will remain unsolved.

In Idaho and Arizona, farms that are properly developed, properly equipped, and properly operated, are paying their way. The families on these lands are enjoying some of the modern conveniences and necessities, and they are contented. Until the unoccupied land is in the same condition, its owners will not be able to pay its portion, and until the proper financing and supervision is available it will continue to be unoccupied.

The class of settler required for this land is no longer willing to undergo the hardships and privations of the old days. Families demand a standard of living in keeping with the times, and if the rural population is to continue to advance in education, etc., and be able to handle its business in competition with the world, there must be modern conveniences on the farm. What manufacturing concern would think of trying to operate without the latest and most efficient machinery and the latest methods to meet competition? Just the same thing should apply to the development of new lands if they are to become a success; modern methods and the best machinery must be used.

The Lethbridge Northern Irrigation District, in Southern Alberta, Canada, would never have been successfully settled had it not been for the policy of supervision and financing put in operation by the Alberta Government. After four years the Government is just beginning to see the results from the application of this policy. It has fully demonstrated, however, that it is "a step in the right direction". In less than four years the district has been fully settled with a desirable class of citizens, most of whom are contented. Fully 75% are in better financial condition than when the project was begun. Most of the settlers placed on the project have stayed. Of 738 families placed, only

26 have moved away. While many problems have been encountered and mistakes have been made, the Alberta experiment has been fully justified. No other agency could have attained the same degree of success.

Nils A. Oslen,[27] Esq. (by letter).—Certain conclusions embodied in Mr. Lippincott's paper were expressed by Secretary of Agriculture Jardine[28] in discussing the significance of the interest exemption on construction charges as a special subsidy to particular locations. He indicated that such a stimulus in principle represents a form of discrimination against established farmers and new settlers elsewhere, and that, in the present period, it is not justified by a need for enlarging the basis for the Nation's supplies of farm products.

At the same time he pointed out, as Mr. Lippincott has done, that reclamation is frequently connected incidentally with other objectives, such as flood control, promotion of navigation, and development of water power. Under these conditions the advisability of a particular project must be determined by consideration of the entire complex of objectives. Even under such conditions, it would be difficult to justify the interest exemption for reclamation.

L. C. Gray,[29] Esq. (by letter).—In connection with Mr. Lippincott's interesting paper, attention may be called to the views of the late Mr. R. P. Teele,[30] formerly of the Bureau of Agricultural Economics, U. S. Department of Agriculture. Mr. Teele had made intensive studies of these matters, and his findings, therefore, have an important bearing on the issue.

The Committee has rendered a public service by its analysis of the Federal Reclamation Policy. Mr. Lippincott might have emphasized still further the points that the exemption of interest (1) increases the local pressure to obtain Federal funds for reclamation projects; (2) tends to stimulate the undertaking of projects which would not be justified by prospective farm income if interest were considered; and (3), therefore, imposes a level of capitalization which must be written down to a lower level when the project is acquired by subsequent purchasers after the period of exemption is concluded.

Frank Adams,[31] M. Am. Soc. C. E. (by letter).—The writer is in hearty agreement with most of the recommendations embodied in the report of the Committee. His remarks will be confined to those recommendations, or parts of recommendations, with which he does not fully agree.

Item (1) in the report holds that the waiving of interest payments to land owners on Government reclamation projects is unwise and that, in the future, Government contributions should appear in the assumption of a part of the cost of project works.

In considering this recommendation one should have clearly in mind that the principal object of Federal reclamation has been to reclaim areas which without Government aid would have remained idle or unproductive. In some cases, it has been the large amount of capital required for the construction of reclamation works which put them beyond the means of private and district enterprise. Again, it has been the excessive unit cost, and the long time required to complete the reclamation through actual settlement and utilization of the land. In still other cases, it was the inability of private or district enterprises to place on the land reclaimed responsibility for paying the cost of the necessary reclamation works. In a few instances, most notably the pending Boulder Canyon development, the Federal Government has been the only agency politically competent to cope with the related interstate and international complications. The Reclamation Act was passed, therefore, not primarily to bring more land into agricultural production, but as an aid to Western development, which, at the time the Act was passed, was lagging. It was very generally accepted as a desirable measure designed to put the opportunities for development in the arid and semi-arid West on a par with those in the East and Middle West.

---

[27] Chf., Bureau of Agricultural Economics, U. S. Dept. of Agriculture, Washington, D. C.

[28] Annual Rept., Secretary of Agriculture, 1927, pp. 26–27.

[29] Prin. Economist in Chg., Div. of Land Economics, U. S. Dept. of Agriculture, Bureau of Agricultural Economics, Washington, D. C.

[30] *Department Bulletin No. 1257*, "Land Reclamation Policies in the United States," also, "The Economics of Land Reclamation."

[31] Prof. of Irrig. Investigations and Practice, Univ. of California, Berkeley, Calif.

In carrying out the National Reclamation Act, it was both desirable and inevitable that the Government should proceed along more comprehensive lines than would have been justified in the case of strictly commercial and district reclamation enterprises. This meant not only added cost in surveys and construction, but also the opening of lands for settlement on such a scale as to involve long delay in their full utilization. In fact, few of the Federal reclamation projects are yet completely settled. The burden of settlement, which has fallen almost wholly on individuals and inadequate local credit agencies, has been a heavy one. Nevertheless, the advantage to many Western States and communities has been tremendous, and the nation has benefited greatly through this process of equalizing opportunities for its people to create homes and develop industries in those parts of the country which, although rich in resources, still lay more or less backward and dormant.

No such reclamation program as has been or is being carried out under the National Reclamation Act would be possible without a large financial contribution by the Federal Government. This has always been recognized and the contribution has been made by waiving interest on the cost of reclamation construction. This procedure has been fully justifiable in view of the purposes to be accomplished. However, the waiving of interest has not been stressed, and, consequently, the actual amount of the Federal contribution has not been known. Therein perhaps lay the chief evil of the system.

There is no question that direct assumption by the Federal Government of that part of the cost of reclamation works which it is to pay, as recommended in the Committee report, is abstractly the more desirable procedure; but will it work? That is very questionable. The writer believes, therefore, that until Congress is ready to face the facts frankly and to adopt direct Government aid, whether in the West, East, North, or South, as conditions may warrant, the present plan of contributions by the waiving of interest should not be abandoned. The work of Federal reclamation has not yet been finished because there are still desirable projects pending which can not be built without Government assistance. The present plan has worked, even if not always entirely satisfactorily, and should be retained until some other plan has been shown to be feasible. The writer is ready to assist in bringing about a change in the public viewpoint, but he is not ready to condemn as unwise the plan that has been responsible for such great benefits as have been accomplished thus far under the National Reclamation Act.

There is another phase of the waiving of interest on the cost of Federal reclamation construction that should be mentioned, for it should be considered when and if the plan of direct Federal assumption of part of the cost, as recommended by the Committee, is put into effect; that is, the need for waiving interest during the period in which settlers on the projects are becoming established. The period during which land is being cleared, leveled, and checked for irrigation, and brought into production, is that in which settlers most need help, but find it most difficult to obtain. The land is not then earning either interest or operating expenses. Lacking help, many of the settlers fail, not only on Federal, but also on commercial and district, projects. The Government could well afford to forego interest on its projects during this period, regardless of whether interest is charged later.

Item (6) of the Committee recommendations holds that the output of power and of water at Federal or State works should be disposed of at wholesale and not at retail. If the Committee would insert the phrase, "in general", the writer could agree. Many public agencies, some of them irrigation districts, have successfully retailed both water and power and are still doing it. It is conceivable that in some cases both the Government and the States will find it desirable to retail the output, perhaps only temporarily, and a policy should not be so crystallized against such procedure as to preclude it when necessary to accomplish the largest benefit, whether to the Government, to the State, or to the consumers. Each case should be decided on its merits and not in advance by doctrine.

Committee Recommendation No. (9) that, except in the case of commitments already made, it is undesirable for the Federal Government to bring new areas under cultivation, coincides with recommendations made by numerous farm organizations, as well as by some of the agricultural economics experts of the Federal Agricultural Department.

The writer does not believe that over-production in certain branches of agriculture and in certain areas of the country at any particular time is necessarily any reason for ceasing the reclamation of new areas if the projects which would reclaim those new areas are themselves economically feasible and desirable because of their contribution to the general welfare of tributary areas. There may be over-development in the country generally at the same time that there is under-development in certain important areas. The test in beginning a new project should be: Is it economically feasible, when viewed in the light of long-time production and price trends, and is it in line with equalizing opportunity as between different sections of the country? Furthermore, can it be settled within a reasonable time? If the answer to either of these questions is in the negative, construction of the project should be postponed.

The Committee probably was led to recommend that no new projects be started at this time by the lack of needed settlers on some of the existing

Federal projects and in many irrigation districts. Admittedly, the demand for land is not active, partly because general agricultural conditions have been unsatisfactory in some parts of the country, and partly because of the small number of settlers with the necessary capital and farm experience to make a success under the generally high costs of present-day projects. However, there is need for Federal aid in some States in connection with supplementing the water supplies of existing enterprises. To cite a single example, California is studying her water resources with a view to the adoption of a comprehensive plan of water utilization. There are areas of surplus water and areas of deficiency, and the cost of equalization will be very high. The benefits to accrue will ultimately justify the expenditure, but there is need of Federal aid in financing, as well as in planning and building the necessary works. Other States have similar problems, and it will require many years for their solving. In the meantime, economic conditions will continue to change and much of the present slack will be taken up.

The writer also desires to take exception to Item (14) in the Committee's recommendations. The first sentence reads:

> "Land settlement, including paternalistic, financial, or any other kind of assistance to the individual farmer, should be treated as a local matter and should therefore be made the concern of the State or locality rather than of the United States."

While firmly believing in the soundness of Governmental land settlement policies that have been successfully put into effect in some other countries, the writer concedes that the present public attitude is generally against those policies for this country. However, if the Government is merely to make existing reclamation projects successful it must do what is necessary to complete their settlement as rapidly as possible. The Government is not justified in asking settlers to go on its projects unless conditions are right for their success.

The Government should at least put the land in the projects into such condition that the new settlers can get a foothold without exhausting their meager capital and being forced to abandon their farms before they are on a profitable basis. The expenditure of $20 to $40, or more, per acre for leveling land and otherwise preparing it for irrigation is not unusual on some of the desert-land reclamation projects. This work could be done far more economically by the Government, working on a large scale with heavy machinery, than if left to the individual farmers. The writer does not advocate having the Government prepare for irrigation all the land on its reclamation projects, but he believes that when the cost of preparation is high and the prospective income per acre is relatively low, both the Government and the individual settlers would be the gainers if the preparation of enough of each farm to permit the immediate planting of perhaps one-half of it, or a minimum of 20 acres, were to be included in the Government reclamation program, the cost of this to be paid, with interest, within a relatively short period after settlement.

This would foster settlement and increase the percentage of individual success by advancing the time when the farms would be producing an income

Failure to obtain a quick income, coupled with the heavy expense of early farm development, and the general lack of local credit, except to those who have substantial resources, is the chief cause of failure of both settlers and projects. Theoretically, the State or some local agency might do this preliminary work which is of so much importance to the settler. Practically, however, it is never done, nor is it likely to be undertaken except by the agency which is most vitally concerned, which, in the case of Government reclamation projects, is the Government itself.

There is still another function which the Government should perform on its reclamation projects but which the Committee might class as "paternalistic". This is organizing the community development and advising as to cultural practices and types of production, the latter being perhaps of most importance when the settler is just starting. Possibly the governmental agency that builds reclamation works is not the one that is best fitted to lead and advise in such matters, although it is not clear that this is the case. At any rate, some governmental agency should be charged with that responsibility, because the Government should not let go as long as its funds are still tied up. The help of local agencies of course should be obtained to the limit, and in some cases such agencies could carry the main part of the burden; but so long as the project is under Federal control, the Government should be privileged to perform such functions as are deemed necessary by those who carry the responsibility for its success. Otherwise, the Government should stay out of reclamation and confine itself to constructing major works, such as the Boulder Canyon, or some of the other large dams built by the Bureau of Reclamation. Real reclamation is not accomplished until the settlers on the project lands are well on their way toward being established.

It is exceedingly desirable that all the worthy Federal reclamation projects should reach the successful stage at the earliest possible date. The Government can afford the loss that will accrue to the Reclamation Fund through delay, but the settlers on the projects cannot, because such projects are now very largely organized as irrigation districts in which all land owners have a joint liability for the construction charges and costs of operation and maintenance. Any competent and solvent business man would supply whatever aid is necessary to complete the success of these projects that can succeed, and there is no justification for the Government not doing as much because of a fear of being "paternalistic".

M. J. Dowd,[32] M. Am. Soc. C. E. (by letter).—It is apparent that the Committee has given considerable thought and study in formulating the principles of "A National Reclamation Policy". The writer is in accord with a majority of these principles, but would like to comment on one or two of them which in his opinion should be questioned.

In considering Principle (1) it should be remembered that the Reclamation Fund is built up from sales of public land and not from taxation. The arid public land is of little value until water and labor are applied. If the original capital costs the Government nothing and it is to the interest of the country that the non-productive valueless land be made productive and if an individual can be found who will set his labor against the free capital which the Government furnishes, it does not seem as if the Government would be justified in making a profit out of the transaction in the way of interest or otherwise. Certainly all the money which comes out of the sale of public land should go back into the development of such land and in so doing the whole Nation profits by the increased wealth. Even where the Reclamation Fund is used to develop land in private ownership, which was formerly public land, the writer believes such land should not pay interest on the capital invested; but in any event, where the development concerns public land alone it seems as if the present policy of not charging interest certainly should apply.

---

[32] Chf. Engr. and Gen. Supt., Imperial Irrig. Dist., Imperial, Calif.

The enforcement of Principle (9) would mean placing an embargo upon agricultural development in the West which is both unwise and unnecessary. The following quotation is taken from a speech of President Coolidge, regarding reclamation in the West:

> "A country growing so fast as is our own cannot limit its considerations to immediate necessities. Many people now living will see this a country of two hundred million inhabitants. The national interest will be served by promoting the widest diffusion of population. The national security, the best social development, the wisest untilization of natural resources, all demand this. Criticism of such a program of encouragement, on the ground that there is already an over-production in agricultural products, lies in the lack of understanding that the projects take many years for development; that they furnish only a small portion of the total increased food supply required even by our increasing population; and that the utilization of their supplies lies in the development of the West itself. It is my purpose, unremittingly, to stimulate and encourage the development of these great projects by every authority of the Federal Government."

The writer believes it is a mistaken idea, as expressed in Principle (9), that reclamation should be curtailed simply because, for the moment, there is an over-production in this country of certain staple commodities. Reclamation projects that are easy of development, have been completed and those that are now being considered are not only complicated but costly, which means that future agricultural development in the West necessarily depends upon reclamation by public agencies.

A reclamation project requires from 5 to 25 years to develop. There are only about 20 000 000 acres of unreclaimed arid land for which there is a water supply available or for which such a supply can be made available. If it were all put to crop as rapidly as possible, it would not be even a serious factor in the National agricultural requirements by 1950. It is estimated that the population then will be at least 150 000 000, requiring production from about 431 000 000 acres in addition to pasture lands. In 1919, there was a total of about 365 000 000 acres in crops harvested. In other words, by 1950 the country will require about 66 000 000 acres more land for crop purposes, with the same yield per acre and the same consumption per person. If all the 20 000 000 remaining irrigable acres in the Western States are put under irrigation, there will yet remain a shortage of 46 000 000 acres

21 years hence. This indicates that, in order simply to keep pace with normal development, there should be reclaimed about 1 000 000 acres per year.

During the period of 1910-20 the population of the country increased about 15% and, in the western part, nearly 35 per cent. These Western States, and particularly California, import a great quantity of their food supply at the present time. California exports virtually nothing in quantity except fresh fruits and vegetables. The coastal cities import from without the State from 10 to 40% of their dairy products.

It is safe to assume that the population of the Western States will increase faster in the future than it has in the past, because of numerous economic conditions, such as climate and the Pacific trade. Therefore, a program of retarded reclamation simply means increased cost of food supplies for the West and a consequent slowing up of western development.

As has already been stated, the carrying out of the policy proposed by this principle means an agricultural embargo on the Western States, leaving the more humid sections of the country to develop as rapidly and as much as they may desire. With approximately 40% of the total area of the United States within the arid States, the placing of such an embargo on agriculture for the benefit of the remaining 60% is gross injustice. Furthermore, the adoption of this principle in no manner affects reclamation of additional areas in either Canada or Mexico.

In the case of Mexico, there has been in recent years a rapid development of reclamation near the American border, and many hundreds of carloads of peas, tomatoes, lettuce, etc., are coming into the United States every year. Because of economic conditions in Mexico such products can be produced cheaper than in this country. With proper protection against competition from this source, there is at present (1929) a market for a considerable increase in the vegetable industry in the United States; but without such protection, even those projects now producing fruits and vegetables will not long be able to compete with the foreign growers. Such protection is economically sound and in accord with the tariff policy of this country.

There is no question as to there being an over-production in certain staple commodities produced in the United States, but this does not hold true for all products. In the Western States, at least in California, there is a shortage of dairy products, hogs, and beef cattle. It has been noted that what often appears to be over-production is simply poor distribution. This applies particularly to the vegetable and fruit industry, where it has been shown many times that the returns to the grower are governed primarily by the manner in which the product is distributed rather than by the supply of the product. Furthermore, there should be considered the rapid increase in the use of certain farm products, such as fruits and vegetables. The saturation point for the orange production was predicted many years ago and yet, because of increased use and co-operative marketing, the returns to the grower are better to-day (1929) than they were ten years ago. As a further illustration, consider the lettuce industry in the Imperial Valley. Shipment from 2 000 acres of lettuce in 1913 flooded the market and the prices received would hardly pay the freight; yet in 1929 about 12 000 carloads were shipped and the net profits are estimated at from \$2 500 000 to \$3 000 000. The same holds true for the cantaloupe industry in Imperial Valley. In 1908, the market could not profitably absorb 1 804 carloads of cantaloupes but, in 1927, more than 17 000 carloads were shipped from Imperial Valley alone and very good returns were received by the growers.

Indications all point to the end of the agricultural depression which followed the World War. Because of this, and the fact that during the intervening period there has been only a slight expansion of reclamation projects, the writer believes the time has arrived for a resumption of the development of Western arid lands.

C. E. GRUNSKY,[33] PAST-PRESIDENT, AM. SOC. C. E. (by letter).—The report of the Committee on a National Reclamation Policy is necessarily more or less of a compromise. Members of the Committee without retracting in any measure the views expressed in the report remain free to amplify their attitude toward the policy therein outlined and will, it is hoped, contribute discussions which may throw much needed light on a problem that has vexed other countries besides the United States.

In line with this thought the writer will refer to several points which should receive somewhat more attention than has been given them in the supporting statement of Chairman Lippincott.

In the first place it seems fundamentally wrong and unwise to make indirect appropriations for the carrying out of public improvements, such as land reclamation. If a project is worthy and deserving of Government aid the appropriation of funds therefor should be a direct appropriation for a specified purpose. Under the Reclamation Act the moneys received from land sales and, in part, from oil royalties drift into a fund which, for a time, was at the disposal of the Secretary of the Interior; since 1914, however, the Secretary's allotments are subject to authorization by Congress. Losing sight of the fact that such moneys would naturally have gone into the United States Treasury and have been there at the disposal of Congress, it was prescribed in the Reclamation Act of 1902 that these moneys should be used by the Secretary of the Interior for the reclamation of arid or semi-arid lands. One-half the money collected in any State should be expended within that State if there were available projects. Due to political exigency, an unnatural demand for the expenditure of funds on work of doubtful merit resulted and this fact may account for the undertaking of some of the earlier projects which should have been let alone. Believing that the proceedings under the law were unsound in principle, unwise, and not in the best interest of the development of the country's natural resources, the writer is opposed to the indirect appropriation and believes that, in this respect, a new policy should be adopted.

In the second place the aid extended by the Federal Government when wet or arid lands are to be reclaimed should not go beyond the regulation of stream flow and the control of floods. The water of rivers and lakes together with underground water is this country's most valuable natural resource. It has been the practice to allow the title to reservoir lands to be acquired by private parties, by districts, and by municipalities without regard to the construction of works in conformity with a comprehensive plan and without Government control of the manipulation of the reservoirs. The regulation of the stream by storage should be such as to produce "the greatest good to the greatest number." Consequently, the people should never allow works for the regulation of the flow of a river to get out of Government control.

---

[33] Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

This does not mean, however, that the United States or the individual State should be the only agency for the actual construction of works. The privilege should be granted to any State, to a group of States, to municipalities, to districts, and even to private parties, always provided, however, that the works conform to a comprehensive scheme of development and that permits shall be conditioned upon use of the storage facilities in conformity with the regulations prescribed in the case of the international, interstate, or navigable streams, by the United States and, in case of other streams, by the State in which they are located.

In the third place the capital investment made for stream regulation and the contribution toward flood control by the United States or by an individual State should not ordinarily be made subject to repayment by the benefited parties. Instead of this practice which prevails under the Reclamation Act there should be delivery of the output of water, or power, as the case may be, at wholesale rates to the district or municipality or private concern which can use it to the greatest advantage. Moreover, when there is a choice of the region in which the output is to be used, due weight should be given to the potential development of the remote sparsely populated sections of the country. In the long run the large development of the outlying regions making for a maximum of decentralization will be of greatest benefit to the nation. Furthermore, commitments should not be of such a character as to curtail this development.

If, under the Reclamation Act, each district served with water had been required to pay for the water delivered to it at a fixed wholesale rate and had been left free to distribute the water to the individual farms, all the trouble in recovering the investment and carrying a project during the development period would have been avoided. The cost of carrying a project during this period, under such a program, would fall on the Government, as it should, and, in that event, this cost could be set down as the Government's contribution to the success of the enterprise.

In the fourth place the owner or original purchaser of lands which are to receive water from a reclamation project, under the Reclamation Act, has received an indirect bonus, which has amounted to 50% or more of the cost of Government works. The giving of a bonus to a farmer or to a group of farmers does not assist farming as an industry. It is unsound in principle and unwise. The prospect of such a bonus immediately adds value to the land, and this value is pocketed by the original owner; it does not benefit his successor. Moreover, while the anticipation of such a bonus by the settler on the lands of the Government project has stimulated farming activities in certain selected localities there has elsewhere been curtailment of agricultural activity.

In recent years more than 30 000 000 acres of farm lands have gone out of cultivation. This, of course, must be ascribed to over-production. Careful consideration should be given, therefore, in the case of every Government-aided land reclamation project, not only to its physical, but also to its economic, aspects, the latter being as much as the former within the province of the investigating engineer.

Thus, for example, the writer, who was early connected with the studies which preceded the carrying out of the Sacramento Valley flood-control project in California, has repeatedly pointed out that the inclusion of Sutter Basin in the reclaimed area was a mistake. This basin, which has an area of about 60 000 acres, lies in a pocket between the Sacramento River and the Feather River. Under natural conditions it was subject to inundation by annual high waters of these rivers. However, a large part of the basin land lies so high that the spring flood waters drain off early enough to permit summer crops to be grown. The State should have purchased all the land in this basin and allowed it to be submerged at river flood stages, thus functioning as a retarding basin. The State would then have received some revenue from this area when farmed to summer crops. Instead of following this course, no study of economic advantages and disadvantages was made. The attempted reclamation has burdened the land with a heavy debt and has also increased materially the cost of the control works in the down-stream reaches of the river, because the withdrawal of the basin from the submerged areas has added to the river's peak discharge at all down-stream points.

In the fifth place land settlement enterprises are local in character and scope, and should never be undertaken by the Federal Government. It should be left to the individual States to organize such enterprises and to make provisions for whatever financial assistance it may be thought wise to extend to the settlers. As already stated, the United States should contribute toward the cost of flood control and, in the case of an output of power or water from the regulation works on an international, interstate, or navigable stream, it should dispose of this output at wholesale, thereby aiding municipalities and districts in securing water supplies, and assuring the success of worthy flood control and other reclamation projects in the several States.

Finally, it is in order to explain why the Engineering Profession must concern itself with such problems as that of shaping the National reclamation policy. The welfare of the country in the years that are to come depends on the utilization and development of its natural resources along well-considered, sane lines. The ordinary citizen is prone to give thought to the immediate situation only. Legislatures are all too frequently driven to conclusions by trading with a hope for local advantage. The engineer, with his mind trained to the careful weighing of every feature which has a bearing on his problem, is pre-eminently qualified to ferret out the fundamental principles which should apply in the conservation of water and other natural resources and to determine how best these principles can be applied. It is the duty of the profession to come forward and give the country the benefit of conclusions reached in this and similar matters. The report of the Committee is timely. While the writer believes, as shown in the foregoing comments, that it does not go far enough, nevertheless it is a "long step in the right direction".

S. T. Harding,[34] **M. Am. Soc. C. E.** (by letter).—This report consists of certain specific conclusions with only limited explanation. The explanatory statement by Mr. Lippincott is in many ways a clearer treatment of the subject than the report itself, as it includes the basis for the conclusions. The difficulty of preparing a report by a large committee representing various points of view on a highly controversial subject is obvious.

In the writer's opinion, the most important element in any policy of public aid to irrigation development is the basis on which such use of public funds may be justified and the extent to which aid may be warranted. The usual arguments advanced by those justifying such aid include the benefits of the added taxable resources, increased crop production, and opportunities of insufficiently financed individuals to obtain farm homes. In its first conclusion the Committee apparently recognizes some public contribution as proper. It recommends that such contribution should be in the form of payment of part of the construction costs rather than as an interest waiver. The report, however, is entirely silent on the policy which should govern in determining the extent of the public contribution or its basis of application in specific cases. If it is a proper public policy to spend public funds in land developments where title accrues to the individual land owners, there should be some basis of policy by which the proper extent of such contribution can be determined. The first conclusion as it is now worded can be accepted by both the advocates of full Government aid and by those opposed to any aid. The former can read into the conclusion a large measure of contribution, the latter practically no aid. In the writer's opinion the Committee has omitted the most important point on which definite conclusions might have been influential in relation to National reclamation policies.

That there is need for some definite policy by which the extent of any public contribution to irrigation development can be measured, is shown by the actions of Congress in regard to such aid in recent years. In December, 1924, Congress authorized the revision of contracts with existing projects on the basis of repaying each year on the construction cost 5% of the average annual gross value per acre of the crops for the preceding ten years. Several contracts were made with existing units on these terms. In March, 1926, such contracts were made with irrigation districts on the Boise Project. The so-called Fact Finders Report[35] gives the average gross crop return per acre for 1913-22, on the Boise Project, as $38.11, of which 5% would be $1.91, and the construction cost as $80 per acre, of which 11% had been repaid up to 1924. The new contract on these terms extended the repayment period from 20 years to about 40 years. The Kittitas Unit of the Yakima Project was adopted for construction by Congress in 1925, on the same basis of repayment. The published reports of the Bureau of Reclamation show an estimated construction charge of $148.75 per acre. The economic report estimated that 96.3 years would be required to repay the construction costs on the "5%

---

[34] Prof. of Irrig., Univ. of California, Berkeley, Calif.

[35] Senate Doc. 92, 68th Cong., 1st Session.

of the crops" basis. In 1926, Congress approved the construction of the Owyhee Project at an estimated construction charge for new lands of $160 per acre, with a 40-year interest-free repayment period. Secretary Work reported to the President that in his opinion the project was feasible and the cost would probably be repaid.

The Boise and Owyhee Projects are located in adjacent areas in Western Idaho and Eastern Oregon. They have similar conditions of elevation, climate, crop adaptability, and transportation to markets. The Boise Project is now relatively fully developed. It represents a typical area of its class in regard to crop returns and ability to pay. If the extension of time for the repayment of an $80 per acre charge on the developed Boise Project from 20 to 40 years was justified by the inability of the land owners to pay in a shorter period, there is an apparently obvious inconsistency in expecting the undeveloped Owyhee Project to "pay out" in 40 years with twice as large a rate of payment per acre per year.

In Conclusion (11) the Committee recommends that interest should be charged on repayment of construction cost and that "the interest rate should be low and the principal payments extended over a long period with no payments on principal during the early years". What is meant by a low interest rate is not stated. A 40-year repayment period without interest is equivalent to paying 2½% interest for 40 years and having the principal written off. Apparently, the Committee had in mind lower interest rates than 2½%, or longer repayment periods than 40 years, if present costs were to be paid in full. If, as recommended in Conclusion (1), the contribution should be in principal and not in interest waiver, with interest charged at 4%, one-half the principal would have to be written off in order to amortize the remainder in 40 years without exceeding the rate of payments for the full cost in 40 years without interest.

Projects such as the Owyhee will have construction costs equal to the present value of the lands to be served after they have been prepared for irrigation. If 4% interest is assumed as a proper rate of return to the Government, based on the terms of the Boulder Canyon Project Act, the Government is making a contribution of $80 per acre to bring about irrigation construction on these lands. This apparently represents present National policies. The Committee in its report does not indicate whether it considers present contributions more or less than are justified.

In his explanation of the report, Mr. Mead states that the measures for aiding settlers in farm development are entirely inadequate, and he recommends either that construction of canals for unimproved lands be stopped or additional aid in settlement be given. If a 40-year repayment period on construction costs without interest is not a sufficient attraction to settlers having the necessary funds to develop the lands, in the writer's opinion, Mr. Mead's recommendation for stopping development for unimproved lands is clearly sound. Neither the Committee nor Mr. Mead has presented any basis justifying a larger contribution than that now effective under the present repayment terms. If this contribution is insufficient to attract settlers with means to develop the lands, it may be an indication that settlers having such means have a clearer judgment regarding the economics of land development than those responsible for such projects. Extension of additional public aid in the settlement of irrigated lands has not proved successful where it has been tried by Western States, either in securing the returns from funds loaned by the State or in satisfaction to the settlers.

The preceding comments are not intended as a criticism of the Committee, as the difficulty of reaching a unanimous report on such a subject is fully appreciated. In the writer's opinion more specific conclusions are desirable, even if the attempt to reach them results in divisions within the Committee with subsequent majority and minority reports.

H. H. DARE,[36] ESQ. (by letter).—The State of New South Wales covers an area of 310 700 sq. miles. Irrigation projects in this State, as well as elsewhere in Australia, are developed by the State, and the Commonwealth Government is not financially interested, except in one case, the Murray River, 1 300 miles long, at the head of which the Hume Storage Dam (2 000 000 acre-ft. capacity) is being constructed (1929) at the joint expense of the Commonwealth and of each of the three States of New South Wales, Victoria, and South Australia, which are watered by the river.

Under the New South Wales Act, which was passed before 1909, the right to the control and use of all water in all rivers and lakes in that State which flow through, past, or are situated within, the land of two or more occupiers, is vested in the Crown. No person is permitted to obstruct the flow, or change the course, of any such stream, or to divert water therefrom, unless a license has first been obtained. Before such a license is granted a public inquiry is held, at which it is possible for any interested person to appear in support of, or opposition to, the granting of the license. It should be noted that the State does not claim ownership in water, but merely controls the streams in the interests of the riparian holders, the license fees collected being sufficient only to cover the cost of administration.

Apart from irrigation of their own lands by landholders, irrigation development in New South Wales is of three classes, namely,

(1) Irrigation Trust Projects, in which the works are constructed by the State, and the cost is repaid by the landholders with interest over a period of years.
(2) Private Irrigation Projects, in which works for the supply of two or more landholders are constructed and paid for by the owner of land fronting a river or lake; and
(3) State Irrigation Projects.

In the case of Classes (1) and (2) the management is local, and the Water Conservation and Irrigation Commission exercises a general supervision only, and makes no advances to settlers. With regard to Class (3), the Commission may initiate and carry on a State Irrigation Project, subject to the provision of the necessary funds by Parliament. The process involves four steps as follows:

(*a*) Acquire all the land within the boundaries of the scheme.
(*b*) Construct and maintain the necessary roads, etc., as well as works for the storage and distribution of water.

---

[36] Commr., Water Conservation and Irrig. Comm., New South Wales, Sydney, New South Wales, Australia.

(*c*) Sub-divide the land into irrigation farms, allot water rights to each farm, and determine the price of such water rights.

(*d*) Make available the blocks for application by means of:

(1) Purchase in fee simple (freehold), with payments including principal and interest at 6½%, extending over 36½ years; or,

(2) Lease (in recent schemes as perpetual leases), with an annual rental as gazetted, for the first twenty-five years and, subsequently, as may be determined to be the fair market annual rental value, irrespective of any improvements.

Concerning Step (*d*), water is either measured at the outlet to each block on a per acre-foot basis; or, in some cases, where meters are not installed, it is charged as a fixed rate per acre per year, covering a specified number of waterings, with an extra charge per acre for special waterings, if required.

Conditions in New South Wales are different from those in the United States, inasmuch as the control of water is vested in the State, and the Commonwealth Government has no authority or interest in irrigation development, except as a fourth partner in the construction of the head storage and locks and weirs on the Murray River. The irrigated lands developed under State enterprise are already owned by the State, or are acquired by it before subdivision.

With regard to the payment of interest on the cost of construction in the case of Trust Projects (Class (1)), which is relatively small, the annual repayments to the State include the payment of interest and those made to a sinking fund to repay the cost of construction over a period of years.

In the case of the Murrumbidgee Scheme, which is the largest of the State Irrigation Projects, the principle was laid down that the cost of the works should not be repaid, but that the interest thereon, together with certain other charges, should be recoverable in the form of rent and water rates. At that time the tenure was entirely by perpetual leasehold, but, with the subsequent change of Government policy, under which conversion to freehold is permitted, the money received from such conversion is allotted partly as interest and partly as principal toward reduction of the capital debt. The values on re-appraisal of the land held under perpetual lease after the expiration of the first period of 25 years and subsequent periods of 20 years, and on conversion to freehold, are fixed by the Land and Valuation Court, if the settlers should make application therefor. Since these values are dependent to a great extent on the marketing position, they may not represent the figure originally estimated for the cost of land plus works. Such difference in values will represent State aid to irrigated agriculture.

In connection with the Murray River, a different principle will probably be followed, inasmuch as there has been no proposal to recover from irrigation settlers in this State the proportion of the cost of construction of the head storage, which is treated as a National work for which the State must pay. The cost of distributary works, however, will be recoverable from the settlers.

It has been argued that free head-works should be a general charge on the community, as in the case of bar harbors and certain other public works, the benefit to the State being realized in terms of new settlers gained and in increased general prosperity. There is no doubt that on the rivers in New South Wales the regulated flow resulting from a head storage is of benefit to all the riparian holders, most of whom do not hold an irrigation block included in any State Project. To apply the principle of "betterment" to these holders would be difficult, and it is not possible under existing law.

The experience in New South Wales in regard to assisting settlers on irrigation farms is in accord with that in the United States, as referred to in Mr. Mead's paper, namely, that:

(1) Due to the increased cost of developing farms, and to the uncertain position as to markets for their produce, there are only a few settlers offering to enter irrigated lands; and

(2) Such settlers as are applying for farms have in very few cases sufficient capital to carry on without assistance. Such assistance is rendered by the State, either through the Water Conservation and Irrigation Commission, or through the rural bank, which raises its money by the same methods as an ordinary trading bank. This money has to be used for rural development purposes, either by way of advances to individual settlers or to co-operative groups.

The bank makes advances on irrigation farms by overdraft on current account, repayable on demand, with interest (at present) at 6½ per cent. Settlers must have the means to effect some improvements before the bank will assist. Advances are limited to certain percentages of the value, depending on the nature of the improvements. Irrigation settlers are not offered more advantageous conditions than those on non-irrigated holdings. Although it is now the policy to have the bank arrange for the financing of irrigation settlers, there are still a large number assisted by the Commission. These include some of the earlier settlers, and also the soldier settlers who took up farms without any capital, in accordance with the land settlement policy adopted at the termination of the World War. Following on the increase in the cost of production, and the fall in market values with regard to some of the products, a re-adjustment is being made in the case of the soldier and, also, of some of the civilian irrigation settlers.

The Coomealla Area, which is the largest irrigation project of the Water Conservation and Irrigation Commission, is situated on the Murray River. It embraces about 110 holdings, each ranging from about 15 to 25 acres, and they are used exclusively for fruit growing, principally vine fruit and citrus. Assistance is given by the Commission by only charging the full water rate at the expiration of five years after the granting of each block. In the intervening years a reduced water rate, progressing from one-fifth of the total upward, is charged. On application by the settler the payments of all monies due for rent or instalment of purchase money and other charges, is suspended for periods extending to 3 or 4 years from the date of granting the block, according to the class of planting, and subject to a charge for interest at 6½ per cent. At the end of this period the amounts thus suspended are compounded, and the settler is then required to enter into negotiations with the rural bank, or to obtain necessary financial assistance otherwise, to pay off the money due the Commission. During the period up to which the debt is taken over by the bank, the Commission makes advances against improvements on the same scale as that allowed by the rural bank, and before any such advances are made a valuation is prepared of the settler's assets, so that at all times the State is in the favorable position of having a secured asset for any money advanced. These advances are also suspended, with interest at 6½%, on the same understanding with regard to their being taken over by the bank at the date fixed.

Blocks were granted to settlers on this area in 1926, and most of the blocks have been allotted in about three years. The settlers have been either the sons of landholders from the surrounding district, or laborers previously employed in irrigation work. In most instances these men have had very limited capital, but they have had experience, and the result has been very satisfactory, because most of the improvements have been carried out expeditiously, and at low cost. The average of the advances made to date (1929) only represent a little more than £100 ($485) per settler. During the 1928 season the production of vine fruit from a number of the blocks has been quite considerable. The bank will be called upon in 1929 to take over the debts of some of the earlier settlers. It is willing to do so because the assets are substantial.

If the marketing position remains satisfactory, the Coomealla Settlement gives promise of being a successful one, but without State aid the position of the majority of the settlers would have been impossible. It has been only by judicious assistance that these men have been able to make good, and, were it not for State assistance, very few of the blocks on the settlement would have been developed. Such assistance, it should be noted, has been made only after a careful investigation of the assets on each block, and after a report from the local manager, who is in close touch with the settlers, and is in a position to form an estimate of the personal qualifications of each. With such safeguards the writer is of the opinion that the principle of State aid is a sound one.

CHARLES R. HEDKE,[37] M. AM. SOC. C. E. (by letter).—An expression of opinion from engineers whose major experiences were obtained in irrigation, on a policy which embraces the basic principles which they now believe should govern, in the future, the National Reclamation Policy, would be valuable and would carry authority and weight in the solution of a very important problem. Such, doubtless, was the aim of the Executive Committee of the Irrigation Division when creating the Committee. Furthermore, to clothe the report offered with the endorsement of the entire Irrigation Division is a worthy effort, but it cannot go unchallenged, because it is felt that many interested members have not been satisfied by the report as a whole. Several of its principles are suspected of error equal to those in the present policy and, what is more serious, the report does not reach far enough to correct the vital and underlying weakness in National reclamation. A most general review of the experiences and lessons from past results seems clearly to justify one viewpoint at least, namely, that the entire problem is much more serious than the curt report of the Committee would indicate. The "Explanatory Statement", issued by the Committee as supplementary, is also not satisfying. It should have preceded the report and should have contained more data and arguments to support some of the principles used.

---

[37] Cons. Engr., San Antonio, Tex.

A policy for a National problem as large as reclamation cannot be framed by stating merely the engineering features involved. Clearly, it must include other and vastly different fields of endeavor. Nevertheless, engineers have been largely in practical control of Government reclamation; they have guided the affairs of that undertaking, and are, therefore, responsible in a very large way for the results obtained.

Somehow, National reclamation has not been an outstanding success in itself. There are many who feel that it should have been successful, and, therefore, several attempts have been made to ascertain the reasons for the developed weaknesses in order to propose changes and remedies. The Special Advisory Committee appointed for the Secretary of the Interior to investigate the results of National reclamation and to recommend changes in the policy gave considerable time and thought to the problem. It proposed corrections which provided a tendency of direction apparently not in line with the opinions of the Committee of the Society. This Society Committee, furthermore, is at variance with the present National policy in some of its original principles.

The Committee of the Irrigation Division doubtless represents a wide range of opinion and is, therefore, entitled to considerable respect. It follows readily, however, that the engineers directly attached to the problem are as well qualified to pass judgment as the Committee. Between these two groups there are apparently some differences of opinion, not as clearly expressed as they might be, and which must be reduced to a clearer understanding before the members of the Irrigation Division can vote intelligently on the report. There is evidence of another group, comprising those who cannot reconcile their experiences with the new principles of the Committee, nor with the present policy of National reclamation. A training that touched the early field of private irrigation enterprises and their financing and grew upon the fundamentals developed from those efforts cannot easily rid itself of certain lessons derived from the successful irrigation developments in the country. Neither can the engineer, with experiences derived from contacts with problems of large financing, reconcile the lack of principles for the security of cost payments under Government reclamation with the statements made of the great values created by these developments.

The following viewpoints and conclusions are offered in defense of the principles underlying private irrigation enterprises. In presenting these in contrast with the principles of the Committee and the Government no spirit of personal criticism is intended, and it is only hoped that some thoughts will be given which may disclose a new application for an old principle, or suggest to the members of the Irrigation Division the deep and serious matters involved in the report. Only the principles that cover the apparent differences between private irrigation developments and Government reclamation will be included.

Principles Underlying Private Irrigation Developments.—The final test as to the feasibility of every reclamation project must lie in the answer to the question: Can it be made to pay? In that answer are involved all the engineering features, all the farm development features, and all the operating features of the project (including its farming), balanced with all the economical soil production features of the entire country. The first three of these can usually be reduced to quite definite figures, representing one side of an equation as costs with profits and losses, while the other side contains the doubtful factors of probable values and returns. The weight of the doubtful side of the equation must still enter the final decision, not as a set of definite figures, but rather as a feeling based on experiences obtained from contacts in the broad field of business. The simplest irrigation developments (those intended for the production of native hay in the mountain valleys) and the most complicated developments of private and Government projects should go through the same steps of investigation to reach the final answer regarding feasibility. The following test question is not unfair and should be kept in mind: Has this practice been followed as thoroughly for Government as for private reclamation?

Farm production costs and the business of farming are not yet as clearly understood among engineers as might be expected. Statistics on the business of farming are easily obtained, which tend to show that it is neither attractive nor profitable. At present (1929), so much is heard about "farm relief" that care is necessary to prevent ideas from becoming distorted and unsound in considering the principles underlying irrigated agriculture. Farming, when considered on the average American business basis, is doubtless not profitable and, what is more important, it has never been profitable in the American sense of the word. Something beyond the strict business of farming has enabled that industry to exist, and even to attract people and grow to its present status in which it has the weight to undertake measures to better its condition.

The present farm relief effort reaches much further and has a much deeper meaning than temporary relief for the farming industry. It is clearly intended to Americanize the industry, and the underlying hope is founded on that vision and faith which has made other American industries grow and prosper. Farming in consequence has a much better prospect now than ever before, and both private and Government reclamation advocates can revise their estimate of the need and can urge underlying irrigated agriculture. However, the economics of the farming industry were the same when the large areas of private irrigation enterprises were built as when the Reclamation Act was passed, and they have continued much the same to the present time. The business of farming cannot be blamed for the difference of success existing between private and Government reclamation. There is something else that is wrong.

Controlling Differences Between Private and Government Reclamation.—The waiver of the interest charge, as provided under the original Reclamation Act, now appears to be blamed, under the principles of the Committee, for the failure of the repayment of costs to the Government. The underlying reason for taking that position must arise from the subsidy feature which the Committee has attached to that financial concession. No discussion is given which justifies its viewpoint, and apparently it was accepted so readily by the Committee that any contrary view might receive scant consideration. Nevertheless, the writer wishes to direct attention to the much more plausible reason which can be given to the "no interest" feature, when it is considered to provide an offset for the financial differences, anticipated as natural and inherited, between Government and private reclamation.

First.—The approach to every private irrigation enterprise is through its ability to create some underlying values and make some money. This always furnishes the urge for the development. The approach for every Government project is clearly through politics. This furnishes the final force which determines the time and place for every Government development.

Politics recognizes but little economic difference between the project with a climatic location of 4 000 day-degrees of heat available for the growing of crops and one with 10 000 day-degrees, or between a project with an estimated cost for water development of $60 per acre and one for $120. There is, nevertheless, a wide economic difference between a project that has 10 000 day-degrees of growing heat available per year under a water cost of $60 per acre and another project that has 4 000 day-degrees under a water cost of $120. Private enterprises have always faced and met, as far as possible, the economic adjustment necessary between the ability of a project to produce and the ability of the project cost to produce. The forces of politics in Government projects superseded the forces of economics in private projects and, in consequence, there was imposed, at the outset, an unequal inherent difference in the ability of the projects to pay.

It was clearly the field of the engineer to equalize this difference as far as possible in the original set-up of the projects, but equalization was apparently disregarded then and its correction is now attempted by concessions and releases. Claims for adjustments of this character underlie the inter-project feeling for an even break from the National Government. There is little to be gained by citing the success of several Government projects in contrast with the many marginal ones. For example, the water users in Montana instinctively feel a grievance against the Government for the handicap imposed on them when their results are so often compared with those of the water users in Arizona and New Mexico. Government reclamation has been treated as one large undertaking, made up of many large units, and there is now no escape from the resulting viewpoint of reasonable equality and justice from the Government.

Second.—The next difference existing between private and Government irrigation involves the bases and principles used for the value and ownership of water.

The principles developed from private irrigation, as reflected by the laws and Court decisions in the arid States, revolved around the viewpoint that water had a distinct and separate value and that ownership was as positive and actual for the water as it was for the land. The principle that ownership of water could only be obtained through its use was much the same as that applied for the acquisition of public lands from the National Government. These principles simply mean that for undeveloped public property, where additional values were created through private effort, ownership could well be given to all. The transformation of the water in a stream into the products from the soil made additional values for both water and land and thereby created the reason for ownership to both. It followed naturally, from private enterprises, that the maximum value of water was obtained from the maximum crop value per acre and not from the greatest number of acres that water could be made to cover. It was a principle established by private enterprise to let the value of the water adjust its economic application.

The practical application of these principles somehow was lost under Government reclamation. The settler on a Government project was given the impression that the payments contracted for by him were for construction only and that they attached to his land. Water value and its ownership were never made clear to him. In early mutual enterprises the fundamentals regarding water became so clear that confusion was never possible thereafter. Private owners did their own work of construction for water, and it then represented a tangible asset of considerable value. Under extended developments, in which some money for construction was borrowed for short periods, water value was still held clear and definite, because the security for the personal notes used was taken on the value of the water more than on the value of the land held by the borrower.

There are sections in this country even to-day (1929) where the collateral for all the usual farm loans is the water stocks available. They are well suited and liquid assets, and they have a ready market because the conditions with which these companies have surrounded their stock make it possible to buy and sell that ownership and transfer its use within reasonable limits. The principle of water value and ownership was carried still further by them under complex stock companies which arranged to borrow considerable money and gave as security a mortgage on their property, works, and water rights, which clearly represented water value. Sound development was made on this basis everywhere in the West.

The first step toward National reclamation, represented by the Carey Act, was based on the principle of water value and is sound fundamentally to-day, no matter what its minor weaknesses may be. The Irrigation District method for water development appeared next; it superseded the National Reclamation Act. It was based on the plan of using the land as additional security for the water loan. Somehow, it now appears that this feature led to the viewpoint that the land was the most important if not the only asset, and, in consequence, much trouble was encountered by irrigation districts. Next appeared National reclamation, by means of which the settler was given the land and the water value was not recognized as any collateral, so there was in reality no security for the cost of the water development. In fact, the entire procedure made it appear to the water user (as he was most unfortunately called) that what he was expected to pay for was a license or a concession charge for the privilege of farming on a Government project. Such certainly was not the intent of the law, and some fundamental principle was clearly overlooked or disregarded to create such a result.

No principle is outlined in the report of the Committee that provides any change toward security of any kind whatever for the repayment for Government cost of water and, therefore, the report has not reached the fundamental source of National reclamation difficulties. The present tendency in State irrigation district laws is more and more toward the personal-ownership aspect for water and value, and if that principle is sound for irrigation districts it may be well to consider it for National reclamation.

Third.—In this section the difference discussed relates to comprehensiveness in private *versus* Government reclamation. It still appears to be the general opinion that the remaining opportunities for irrigation developments in this country are too large; that the cost repayments are impractical for private enterprise to undertake; and that the National Government alone can do this work. Within the meaning of private enterprises, which is taken as everything other than National and State developments, it should be remembered that about 90% of the total irrigation of the country has been accomplished. The viewpoint controlling comprehensiveness, as provided from the experiences and results of private work, is quite different from that used for National reclamation.

Private enterprises first made use of the water from the normal flow of the streams. When this flow became insufficient a supplemental supply was provided by small reservoirs which were filled from the flood flow. When the practical limit of diversions from the average seasonal flood flow was reached, the next step consisted in providing a large reservoir, situated in the bed of the stream, which was used for the hold-over supply from large to small run-off years. In that manner the entire available water supply was developed under the continual influence of the demand for more water by having additional land directly available. The period intervening between the use of normal stream flow and the creation of large reservoirs has been a long one in private developments.

In the meantime, a most important result of irrigation was disclosed. Return waters, obtained naturally or from drainage, were found to be considerable and their re-use introduced another stage in comprehensiveness. The private method for stream development consisted of four distinct stages; took many years to complete; was financially sound for each step, since the method itself created the demand for the water; and the final costs were less than under the one-stage Government reclamation method. Comprehensive development was actually reached through the four-stage method; it has been far from complete in the case of the Government method.

Comprehensiveness in Government developments clearly consisted in reaching for the limit of the physical features involved in each project at the outset. No hesitancy is apparent on the part of the Government in going as far as possible: (*a*) For size of storage attached; (*b*) for size of distribution provided; (*c*) for permanency of construction furnished; and (*d*) for such other features which appealed to the public of the section, equalized the expenditures between the benefited States, and involved large sums of Government money. No attempt was made to start anything smaller which would grow larger as financial strength was created by water demand from sound farm developments. The basis used for the physical set-up was wholly in keeping with all Government undertakings, and there resulted a depressing effect because the projects never became economically adjusted between water supply and demand.

This uneconomical feature in Government comprehensiveness resulted also in depressing private irrigation which was directly discouraged by the large water filings made and embargoes imposed. A correction for this condition appears in the principle now generally advocated which provides that the Government undertake only the construction of the controlling feature in any stream development, such as hold-over reservoirs for flood control, power, and water supply, and that water be sold thereunder to all approved users when and as the demand exists, whether they are municipal, irrigation, or power organizations, whether they are large or small, and whether the projects are existing developments or new ones. The concept for Government comprehensiveness appears to be changing to Government conservation; therefore, some unwise feature must have been operating on the former basis.

Fourth.—Another difference exists between private and Government viewpoints in regard to the aspect of land inclusions under a project. This affects directly the marginal lands. Because of their topography, soil, economic development costs, or economic location in the project, these lands are seen to be unable to stand the cost of the water-supply burden by the owner or settler, at least during the beginning period of the development. The existence of such marginal lands under Government reclamation is now doubtless indicated by the 28% of the land idle on the projects. They clearly represent a heavy dead load. The reduction of this load has apparently been slow and difficult and has created the major problem for the present Reclamation Commissioner.

Managers of private irrigation enterprises fundamentally base their unit costs on the unit of water supply available or developed. They may convert that figure to an acreage basis, but acreage is never the controlling factor in their computations. Their projects have grown strong and sound on the water-value principle, and its practical application has provided the natural and automatic exclusion of all lands not suitable to stand the cost of the water burden. Water is the commodity product of private irrigation, and its use has been allowed where and in those quantities that best suit the buyer of the product. The market for the product is considered carefully and is always one of the controlling features in determining feasibility.

Government reclamation definitely attached the water to the land. Wherever this attachment was suspected of being unable to transform the potential values of both water and land into actual values, sufficient to make them attractive, both have remained unused and, in consequence, the attachment seems unsound. Fixing the place of use for water by any one other than the party paying for it, is unwise according to private irrigation principles. Water and land must go hand in hand, each assisting the other; it takes both, used jointly, to transform their potential values into actual values. Increments in the value of each are the objectives of private irrigation.

Attaching the entire increment of value created by irrigation, whether potential or actual, to the land is unsound according to private irrigation experience. Land speculation simply consists of attaching an unearned increment of value, due to possible irrigation, with some or even all of the water value, to the land. Speculation was really aided under Government reclamation by having the value of the water unencumbered, because it was unrecognized, and therefore it was easily attached to the land by others as an unearned land value. Land speculation has never been serious in places where there was any fair excess of land to which water could be applied and on which it must be used in order to get any substantial increment of value.

No unsound colonization is possible when the provision is made that the entire increment of water value (due to its use and above its construction cost) is arranged to accrue to the user of the water. The best control for sound development, as seen from the lessons in private enterprises, is to have available considerably more land than the available water can possibly cover, with the further condition that the right to the use of the water will accrue to the first in time of use and payment of cost. A principle which would provide a sound economic relation between the areas of land and the water available for Government projects will correct much of the weakness inherent in such projects.

Fifth.—The standard of engineering for the construction of Government reclamation projects is now generally accepted as the standard of the country. Credit and praise must be accorded the Engineering Profession, and the Reclamation engineers in particular, for the character of the works built. Substantial and lasting benefit has resulted to private irrigation from the problems of design and construction solved under this development. However, engineers in private practice have never been able to dovetail all the Reclamation standards into any one project and fit them into a cost analysis acceptable to the owners and backers of private enterprises. The Reclamation standards are not in line with the experiences and results obtained in private developments. Private enterprises must obtain their initial water cost as low as possible because (1) margins are computed as more than this cost; (2) the prevailing interest charge equals the original cost in about fifteen years; (3) the true direction and tendency of the best suited system of cropping for the area are disclosed in about fifteen years; and (4) because a value will be created for the re-use of return waters sufficient to pay for the cost of the drainage. These four reasons all converge for a re-adjustment of the construction and financing of a project at some later period and hence the permanency of construction can be made more suitable and economical at that time than can ever be anticipated in the beginning.

The principle that the permanency of construction shall more nearly parallel the values created by its use has resulted from private developments. This is clearly in line with all private developments and, more particularly, with farm developments. Confusion should not be made between permanency and safety. National reclamation inherited a high standard for its construction just because it was National Government work. No other basis was possible and the statesmen responsible for the Reclamation Act doubtless knew such would be the case. The time-honored standards for all Government work were imposed on National irrigation at the outset, and a burden thereby resulted that was not attached to similar private enterprises. To justify and offset that inevitable condition may be one of the reasons why the law-makers provided for the waiver of the interest charge.

The principles that create the differences between private and Government irrigation enterprises may be summarized in ten items. Private developments provide that:

(1) The broad foundation between the natural and economic forces involved should be made sound.
(2) Water is a National natural resource like land and should be treated in all economic and legal aspects as being similar to land.
(3) Comprehensiveness is best obtained by separate steps, each of which should be made financially sound before the next is taken.
(4) Because there is much more land than water available in the arid regions, conditions should be provided that only the best lands, under an economical project set-up, will ultimately secure the use of the water.
(5) The permanency of construction should be made parallel to the values of the water they create.

In contra-distinction to Principles (1) to (5), the following refer to National irrigation, which uses:

(6) Political expediency for projects, instead of economics.
(7) The "forever public" ownership of water basis, in its security aspect for water value.
(8) National comprehensiveness, instead of public conservation.
(9) The inseparable relation of public water to land, in its appropriation basis.
(10) Permanency of construction in line with other Government construction, in its work of reclamation.

It is apparent from Principles (6) to (10), inclusive, that: (1) Most of the fundamental and controlling forces underlying National reclamation are quite natural and inherited; (2) they all operate quite differently from those underlying private enterprises (Principles (1) to (5)); (3) they all circumscribe results and add burdens; and (4) undoubtedly the statesmen responsible for the Reclamation Act instinctively felt these differences and knew their financial effect so that they met it, in the best manner possible, by waiving the interest charges. It appears to be a strong case of attempted equalization and adjustment and nothing like a subsidy in any possible sense. It seems to be entirely wrong to change that interest waiver feature until the principles underlying National reclamation are changed. Viewing that feature in this manner leaves no basis for an alleged subsidy and consequent paternalism stressed by the Committee. In fact, a thorough analysis of the inter-project value for the financial offset through the interest waiver feature may disclose more inequalities and may show that some projects are entitled to even greater concessions before they are on a par with each other and with private enterprises. This much seems certain: Whatever waivers, releases, and concessions have been given thus far to the various projects, they were well founded and were justified under the inherent conditions attaching to National reclamation.

The interest-offset feature in the present policy is still wise and will remain so until a better understanding is had of its true meaning. The principles of protection and correction are introduced and admitted by this feature, and it will remain such until a sound economic adjustment is made of the whole problem. It is the wisest feature in National reclamation because it will some day straighten out forever the absolute foundation of irrigation, namely, the economic and legal aspect of water. It does not subsidize the remainder of agriculture for the benefit of the few who have been experimented upon for National reclamation. The Committee has not reached the underlying weakness in National reclamation by its principles for a new policy.

It may be well to return to the subject of irrigation districts. Organized under State laws, these districts have been given a quasi-public character and, in consequence, their leaders have had a tendency to follow along the lines of National reclamation in the matters of value and ownership of water. Furthermore, they have been supervised by the States which introduced the features of water conservation, project comprehensiveness, and construction permanency, along the lines of the National policy. They were then, through necessity, so set up and "geared" that a burden was imposed on the present to conserve and provide something for the future. Wherever this burden has been too great the irrigation districts have had difficulties. Wherever an offset feature was provided to relieve those imposed conditions the difficulties have been less. The waiving of the interest charge was not possible nor practical for them, but, nevertheless, some other offset feature, having the same result, must be introduced and made effective in order to insure that such projects may become sound and feasible.

This offset feature may take the form of an income from a hydro-electric by-product wherever possible, or as revenue from the taxation of property that has been benefited by the development, and doubtless other ways and means will present themselves. The whole field of irrigation district experience has made it clear that some offset must be provided wherever any tax is imposed for requirements beyond the immediate and strictest reclamation needs of the new settler. New irrigated agricultural developments alone cannot stand the burden of conservation, comprehensiveness, and permanency, and where these burdens have been added to reclamation the entire experience therefrom shows it is unwise to attempt it. The interwoven character of the farming industry with the other industries of the country makes it reasonable to consider that even the entire industry should not alone stand that burden. It appears that there should be some principles in the report designed definitely to allocate burdens, if Principles (8) and (10) are considered essential.

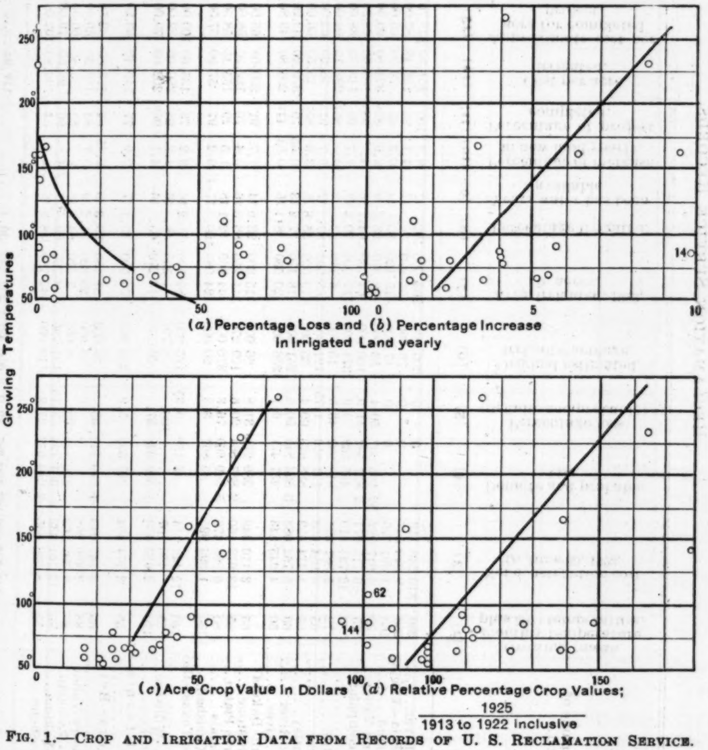
The National Government rightly and naturally should assume the original ownership and control of the natural resource of water. Some of that resource should rightly and naturally be then turned over to the individual States, and it follows readily that some of it should be made inter-State. When, however, the character of that resource becomes so complex as to involve many States, and its control is so interwoven that responsibilities become unnatural, even for inter-State relationship, then the National Government must exercise its parent duty and step in to meet the problems involved. The plan for Mississippi River control is then clearly National and is primarily intended for protection to large property values and many people. The Boulder Dam situation is still a project, while the control of the Mississippi River is a problem, and Congress evidently has taken that view of them. The principles suggested for the control of floods offered by the Committee are not sufficiently clear to provide a basis for the allocation of the costs entailed, and it is suspected a considerable problem is involved therein.

The stressing of the economic aspect of the agricultural industry seems out of proportion in the report of the Committee. The cropped area of Government irrigation, in a quarter of a century of effort, represents less than 0.4% of the cropped area of the country. The idle lands mentioned by the Committee as existing in the Rocky Mountain area and along the Pacific Coast are doubtless nothing more than the marginal lands which have been found unprofitable under the changing conditions of supply and demand through the general development of the country. That process of elimination has been going on for a long time and will continue, because lands which once were profitable only on account of their location to markets, have become unprofitable under the extension of the transportation systems of the country. Every agricultural area is now able to compete effectively with every other area, as far as its availability to markets is concerned, and, therefore, the profit problem is resolving itself more and more into determining crop adaptability, or the ability of Nature to produce. Consequently, some areas are certain to become idle when faced with fundamentally adverse conditions.

Adjustments of like character have gone on in every industry and must be faced in agriculture. There is certainly a benefit to an industry to furnish conditions in which it is possible to produce output values 2.5 times as much as the average. It is a significant fact that irrigated agriculture is in the lead for value of gross production per acre in the industry. It is, therefore, the soundest basis for farming in the country and so, if any curtailment is made, it is clearly in error to place it there. Land settlement is becoming more and more a problem of creating advantages in one location over that in another, for the people already in the farming business. The arrival of a new settler, on an irrigated project, does not necessarily mean that he is a new farmer.

In conclusion, it is evident that National reclamation is still an experiment. It is apparent that every one concerned with the problem, including the land owners on Government projects, desires changes. Some of the suggestions offered are so fundamental that a wide departure from the present policy is involved. It would seem to be a sound beginning policy to undertake correction of existing projects first. The act of postponing all new projects might develop some weight for the step, but this should clearly be understood to be for no other reason than first correcting wrong tendencies in the present policy. Correction seems a case for education, involving so many phases of the problem that it must start at the bottom, where it is believed the most difficult part of the problem lies. Engineers have had so much to do with this Government undertaking that they can well afford to busy themselves to bring about the first changes. Congress has been lenient, but not complimentary toward the results obtained. That unwise things have been done to date is no serious criticism. The principle of waiving of interest charges is considered and agreed to be an unsound financial policy and should be among the first of those corrected. However, merely to remove that feature, simply because some one considered it a subsidy, will not correct the present weakness in National reclamation. Finally, that feature is luckily in the present Reclamation Act and policy of Congress and is considered to represent a far-reaching principle of correction, wisely introduced by real statesmen, for the missteps in the march of progress, and, therefore, in it now lies the greatest hope for the solution of the fundamental problems involved.

Charles Kirby Fox,[38] M. Am. Soc. C. E. (by letter).—Numerous timely and important papers on irrigation and related subjects have appeared in *Proceedings,* but for some reason the climatic conditions and the relation between the acreage cost of the system and the gross acreage crop values seem to have been overlooked.



(*a*) Percentage Loss and (*b*) Percentage Increase in Irrigated Land yearly

(*c*) Acre Crop Value in Dollars (*d*) Relative Percentage Crop Values; $\frac{1925}{1913 \text{ to } 1922 \text{ inclusive}}$

FIG. 1.—CROP AND IRRIGATION DATA FROM RECORDS OF U. S. RECLAMATION SERVICE.

Some time ago the Reclamation Service published data relating to the costs, crop values, etc., of its projects. These data are available for quite a period of years, over the entire West, and presumably are comparable, since

---

[38] Cons. Engr., Los Angeles, Calif.

TABLE 1.—COMPILATION OF CROP AND IRRIGATION DATA FROM THE UNITED STATES RECLAMATION SERVICE RECORDS.

Project No.	State and name of project.	Growing (mean monthly temperature plus 50°) temperature.	Net construction cost to June 30, 1923.	Definite and probable loss.	Percentage loss (definite and probable).	Original estimated irrigable acreage.	Area irrigated, 1922, in acres.	Percentage irrigated.	Years water has been available.	Percentage of increase in new land yearly.	Percentage of project completed.	Cost per acre irrigated.	Approximate cost per acre for completed project.	Yearly maintenance cost per acre irrigated.	Number of water users, 1922.	1918–22 (10-year), average crop value per acre.	Percentage of crop value to cost per acre, irrigated.	Percentage of crop value to final acreage costs.	1925 crop value per acre.	Percentage of $\frac{1925}{1918\text{-}22}$ crop values.
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
1	Arizona—Salt River........	230	$10 548 119	0	0	200 000	203 300	102	17–5	8.5	100	$51.90	$48.70	......	4 250	$62.37	121	128	$103.05	163
2	Arizona-California—Yuma.	260	9 026 547	0	0	76 966	53 970	70	17	4.1	95	167.45	83.00	$4.94	1 211	74.00	44	89	84.56	114
3	California—Orland.........	160	1 091 796	0	0	17 000	15 120	89	14	6.3	85	72.20	60.00	3.08	663	48.95	68	82	44.55	91
4	Colorado—Grand Valley....	92	4 017 922	$2 105 037	50.4	53 000	12 370	23.3	9	2.6	82[4]	324.80	90.50	4.40	389	48.74	15	54	52.89	108
5	Colorado—Umcompahgre..	75	6 715 074	2 801 582	41.9	100 000	64 780	64.7	16	4.	98[4]	103.60	68.50	2.12	8 639	44.56	43	65	49.47	112
6	Idaho—Boise...	80	12 731 410	495 369	3.9	371 700	112 000	30.2	18	1.7	99	113.70	38.00	2.80	2 686	40.26	35	106	35.47	88
7	Idaho—King Hill...........	72	1 881 391	818 982	43.5	15 000	6 440	43.	3	14.	85[7]	292.15	104.00	6.60	160	39.21	13	38	30.76	78.3
8	Idaho—Minidoka...........	65	8 054 663	141 959	1.75	121 000	105 590	87.	17	5.1	91[8]	76.20	62.00	0.93	2 454	37.07	49	60	51.40	138.2
9	Montana—Huntley.........	65	1 474 409	888 623	60.4	35 000	19 520	55.8	16	3.5	89[7]	75.50	50.00	2.46	578	31.72	42	63	38.57	123
10	Montana—Milk River.......	60	6 762 083	3 824 845	56.6	251 906	18 170	7.2	13	0.55	79	372.20	43.20	4.45	178	16.80	5	39	15.	89
11	Montana—Sun River........	50	4 245 843	257 471	6.1	256 000	20 530	8.1	15	0.54	44[7]	206.80	49.80	1.83	877	20.92	10	42	20.25	97
12	Montana-North Dakota—Lower Yellowstone......	65	3 110 449	989 271	31.8	66 000	15 600	23.6	14	1.7	91	200.00	65.00	1.82	572	24.75	12	38	35.14	142
13	Nebraska—North Platte....	65	13 672 160	2 887 864	20.6	129 270	111 250	86.5	16	5.4	70	122.90	158.00	1.71	1 685	30.00	23	19	31.91	106
14	Nevada—Newlands.........	80	6 988 476	5 349 660	76.3	370 000	44 960	12.1	18	1.5	49	155.45	58.50	2.42	788	25.27	17	44	27.85	110
15	New Mexico—Carlsbad.....	165	1 393 995	45 867	3.3	20 000	24 080	120.	17–4	9.2	97	57.90	57.50	2.40	474	45.00	77	77	62.69	139
16	New Mexico-Texas—Rio Grande..................	140	12 146 114	43 158	0.35	175 000	89 590	51.2	16	3.2	73	135.60	100.00	3.00	3 204	48.18	36	48	87.65	182
17	North Dakota—Williston...	65	460 107	....	100.	8 797	1 590	18.	16	1.2	64	290.00	82.00	17.80	76	29.22	10	36	.....	.....
18	Oregon—Umatillo..........	110	2 683 400	1 539 028	57.	20 140	13 270	65.	16	4.1	96[9]	202.00	79.00	3.20	544	45.51	23	58	28.32	62.2
19	Oregon-California—Klamath................	55	3 974 464	233 395	5.9	236 401	36 000	15.1	17	0.9	77	110.40	33.20	1.67	412	20.31	18	61	20.13	99
20	South Dakota—Belle Fourche..................	65	3 547 045	1 279 139	36.	79 000	31 150	89.3	16	2.45	84	113.85	42.50	1.27	983	17.14	15	40	16.78	98
21	Utah—Strawberry Valley..	88	3 466 968	0	0	60 000	30 820	51.2	9	5.7	84	112.50	69.30	0.80	2 400	53.96	48	78	43.32	80
22	Washington—Okanogan...	85	1 302 162	820 173	63.	8 650	5 570	64.5	16	4.	85	238.80	160.00	6.95	493	144.25	62	90	214.18	149
23	Washington—Yakima......	85	12 161 932	381 192	3.15	181 769	123 700	68.	17	4.	96	98.30	79.00	2.00	4 365	93.21	96	107	118.18	127
24	Wyoming—Shoshone.......	55	8 290 865	2 249 672	27.2	110 000	42 780	88.8	16	2.4	54	193.80	102.00	1.60	935	25.96	18	25	25.61	99
	Total, or average......	....	141 787 006	27 102 287	19.0	3 056 427	1 202 180	39.4	....	.....	....	117.94	.....	......	31 468	47.50	....	....	61.65	130

they were compiled by one organization. The Reclamation Service Fact Finding Commission compiled data relating to the losses, etc. In Table 1, the writer has approximated the mean monthly temperatures from the U. S. Weather Bureau records (see Column (2)). The values in Columns (11) and (15) are from the annual reports of the U. S. Reclamation Service. Columns (3), (6), (7), (9), (12), and (14) contain data from the *Reclamation Record*[39] and the data in Columns (4) and (19) are from the *New Reclamation Era.*[40]

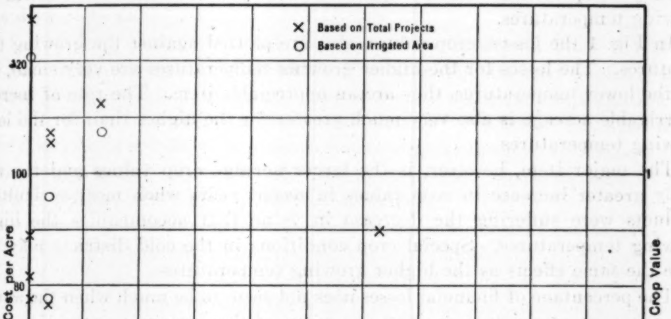


FIG. 2.—RELATION BETWEEN CROP VALUES AND LOSSES ON IRRIGATION PROJECTS, U. S. RECLAMATION SERVICE.

The remainder are computed as follows: Column (5) $= \dfrac{\text{Column (4)}}{\text{Column (3)}}$; Column (8) $= \dfrac{\text{Column (7)}}{\text{Column (6)}}$; Column (10) $= \dfrac{\text{Column (8)}}{\text{Column (9)}}$; Column (13) =

---

[39] *Reclamation Record,* December, 1923, p. 314, and August, 1923, p. 274.

[40] *New Reclamation Era,* February, 1926, p. 18.

$$\frac{\text{Column (3)}}{\text{Column (11)} \times \text{Column (6)}}\text{; Column (17)} = \frac{\text{Column (16)}}{\text{Column (12)}}\text{; Column (18)} = \frac{\text{Column (16)}}{\text{Column (13)}}\text{; and, Column (20)} = \frac{\text{Column (19)}}{\text{Column (16)}}.$$

Most crops start to grow when the mean monthly temperatures range from 42 to 48° Fahr., but the growth at the lower temperatures is slow and for that reason 50° Fahr. was selected as the base (see Fig. 1). The cumulative mean monthly temperatures were computed above that base and were called the growing temperatures.

In Fig. 1 the losses, crop values, etc., are plotted against the growing temperatures. The losses for the higher growing temperatures are very small, but, for the lower temperatures, they are an appreciable item. The rate of increase in irrigable acreage is also very much greater for the higher than for the lower growing temperatures.

The major item, however, is the larger acreage crop values and the relatively greater increase in crop values in recent years when most agricultural products were suffering the decrease in value that accompanies the higher growing temperatures. Special crop conditions in the cold districts may produce the same effects as the higher growing temperatures.

The percentage of financial losses does not seem to be much when the annual gross acreage crop values are 50%, or more, of the acreage construction costs, but on projects costing three or more times the crop value the percentage of loss seems to increase as does the relative cost (see Fig. 2).

RHODES E. RULE,[41] ASSOC. M. AM. SOC. C. E. (by letter).—It is indeed unfortunate that the history of irrigation in the United States is marked by so many financial disasters. The fact that "26% of all irrigation districts organized have failed in some period of their development" is evidence that these projects have been undertaken with an enthusiasm blind to economic considerations.

The percentage is not so great compared to the failures in private enterprises, such as banks and industrial projects, but, unfortunately, the losses in the majority of these projects have been incurred by the purchasers of district bonds rather than by the enthusiastic promoters.

In California, the early history of irrigation district enterprises was so replete with financial failures that many bankers and financiers of the old school have built up a resistance to projects of this character that has worked a great hardship on even the most promising and substantial of the more recently formed districts, developed under the most rigid and intelligent State supervision. In districts having most favorable conditions of water supply, soil, climate, transportation, and demand for lands by legitimate settlers the bonds have had to be sold at discounts which have made the actual interest rates paid by the districts as high as 7 or 8 per cent. Nevertheless, despite the difficulties of financing and the high interest rates that these districts have been forced to pay there has been a tremendous expansion of the irrigated areas in

---

[41] Civ. Engr. (Rule & Rule), San Diego, Calif.

California almost entirely under the irrigation district method. Between 1912 and 1926, one hundred irrigation districts were formed with a total bonded debt of $120 000 000. The total area involved was 3 500 000 acres, which is greater than all the Federal projects combined. These districts have met their financial obligations as they came due in almost every instance.

The reasons for the success of these projects can be summarized into the following items: (1) Adequate water supply; (2) favorable climate; (3) good soils; (4) good marketing conditions; and (5) a bonded debt within the ability of the district to pay.

Before an irrigation district can issue bonds the California Bond Certification Commission, consisting of the State Engineer, the Attorney General, and the State Superintendent of Banks, makes a comprehensive investigation to assure that the district meets these five conditions. The result has been that all the districts have had, to a greater or less degree, the one vital characteristic necessary to the success of any irrigation project—attractiveness to the adequately financed, intelligent, industrious farmer.

In comparison with reclamation projects in the interior arid States, projects in California have two environmental advantages, namely, an equable climate and proximity to great population centers and their attendant cultural advantages. These are of utmost importance in considering the colonization feature of a project. In determining the feasibility of a project that lacks these features the other conditions must be so favorable that they offset the disadvantages of an intensely hot climate and remoteness from large cities. The opportunity for financial gain must be greater than that in a more pleasing environment. For this reason, conditions for marketing agricultural products must be exceptionally good to induce settlers to take up land in the reclamation projects of the desert region.

That settlers will suffer unpleasant environmental conditions in order to take advantage of the opportunity of large profits is demonstrated by experiment in the Imperial Valley. In a period of twenty-nine years, this region has been converted from a forbidding desert into one of the most productive and richest agricultural sections in the country, with a population of 60 000 on an irrigated area of 525 000 acres. This growth has been made without subsidies of any kind.

It is probable that many of the Reclamation Service projects that have been in difficulty would have been more successful if a somewhat different policy had been adopted. A settler entering upon the raw land has a great and sometimes hopeless task confronting him. The preparation of land for irrigation, building of home and barns, purchase of farming equipment and stock, learning new methods of farming necessary under local conditions, building of roads, schools, churches, and all the service utilities necessary to a prosperous community, all require capital, intelligence, and an indomitable courage that the average land seeker does not possess. Unfortunately, too many of the settlers on reclamation projects are those who have failed in their previous endeavors and are seeking new opportunities in a new land. In most cases the causes of their original failures are still present in their new environment and they either abandon their efforts or hold on by a thread without contributing any support toward building up the wealth of the community. The burden of development is on the few industrious and, in most cases, adequately financed pioneers who understand farming and are fully aware of all the difficulties that must be overcome.

If conditions were such that a larger number of the latter type could be attracted to settle on the land, development would be more rapid and failures fewer. If the amount of original capital required for a settler to establish land on an earning basis were less, a larger number of settlers of the right type would be attracted to it. This might be accomplished by including in the project construction some things that now are accomplished by the expenditure of cash or labor on the part of the settler. As Mr. Mead suggests, this might include leveling of land and the construction of roads. Community buildings and utilities might also be included in some instances. Any steps along these lines should be taken with caution and after due consideration of all factors. A small scale experiment should be instituted before great expenditures of the public money are made. The experiences of some Utopian colonization schemes should not be forgotten.

As suggested by the Committee, subsidy by the Government should take the form of an assumption of a part of the total cost of the project rather than aid to the individual farmer. Local subsidy should be largely of a similar nature. Regarding the latter, there is no question in the writer's mind that, in many instances, such measures are justified. The development of new agricultural areas creates wealth and affords means of livelihood for the inhabitants of the towns and cities. That a State or county is justified in lending aid to reclamation projects within its limits cannot be denied. Another form of assessment against those indirectly benefited is the inclusion, within an irrigation district, of cities and towns lying within the irrigated area and the taxing of these lands by the district to pay the interest and principal on a part of the bonded debt. In many cases these cities and towns would not exist if it were not for the irrigation projects, and virtually all business in them can be traced indirectly to the agricultural back country. Surely a part of the cost of the project should be borne by these lands.

George M. Bacon,[42] M. Am. Soc. C. E. (by letter).—There seems to be a tendency to regard the report on a National reclamation policy as inimical to the U. S. Bureau of Reclamation. It would be very unfortunate if implications were read into this report which did not really exist.

The supplementary remarks by Mr. Mead comprise a well-reasoned argument for aid by the Bureau of Reclamation to the prospective settler on present and future projects. As a general proposition there should be no quarrel over this; on the other hand, if the Bureau should take over this work—and it seems to be the proper agency—it would still form only a small part of that in which it can be rightfully engaged.

In the West, engineers are rather familiar with the conditions that have caused some of the projects to be, for the present at least, failures from a financial point of view. The fact of such failures is not vital and it would be difficult to find any enterprise involving the expenditure of $125 000 000 in which mistakes of considerable size were not made. The immediate and proper question is the avoidance of similar mistakes in the future and there is much doubt in the minds of the Committee whether such steps are being taken.

---

[42] State Engr., Salt Lake City, Utah.

The fundamental idea in setting up the Reclamation Fund was that it should be a revolving fund and that expenditures from it should be repaid. This idea has been departed from to the extent of at least $14 000 000, according to Mr. Mead's figures, and there is no question as to this being an entering wedge for further departures. There are many who argue that the indirect benefits derived from construction of the various projects more than outweigh the losses, but it seems to the writer that this argument does not go to the root of the situation. If the construction of projects which do not pay is to be gone into for their indirect benefits, it should be definitely understood before the project is started. It may possibly be that the funds for Federal reclamation should be spent for indirect benefits, but it is obvious that any policy of that kind, or tendency in that direction, is extremely dangerous. Should it be tolerated it would mean that projects were built for sentimental reasons, or on account of political pressure, and it might be pointed out that the pressure for the construction of the various projects almost never comes from those who have to pay the bill. The prominent advocates of new projects to bring in raw land will be found to be, not the farmer, but various urban interests and organizations.

The Committee sought to stage lines upon which the work of the Bureau might be continued with benefit to all, keeping within the fundamental limits set by the establishment of a revolving fund, and that its report seemed to be critical as to some of the work done by the Bureau should not be misunderstood.

It should be noted that for years the Bureau has been limited to a large degree in its choice of projects. The law now requires a specific appropriation by Congress for the various projects and most people know how appropriations are usually obtained. The writer feels that the Bureau could be safely trusted to pass on the economic merits of a proposed project and, with the understanding that a project must pay, limit the construction of new projects to such as are economically feasible. There is no question in the minds of engineers competent to judge that at least two projects are being constructed at the present time which will not pay in this sense and others of this nature are not far distant.

The danger of Mr. Mead's plea for help to the prospective settler is that new projects which will not pay, may be started with a tacit understanding that, if anything does happen, a considerable portion of the cost will be written off and then various aids will be offered to prospective settlers. Following this line of procedure one step further, the day may come when the money furnished to aid the settler need not be wholly repaid and then the argument can again be advanced that the indirect benefits warrant such a course.

There are signs of a growing belief among careful students that from now on history, as to growth of total population, or rural population, will not repeat itself in this country. It is even believed that the total agricultural area under cultivation at present is ample to provide for the needs of the country for a long time. Speaking of the arid and semi-arid West there is no question at all that the total area under cultivation is capable of great increase in produce once its present water supply is supplemented. There are many thousands of acres of land on which production is curtailed because the water supply falls short before the end of the crop season.

The policy in all ordinary business is to bring the plant up to full capacity before enlargement but, as far as agriculture is concerned, the exact opposite has been, and still is, strongly advocated. It is very difficult to realize the unconscious pressure which is being exerted all the time in favor of an increase in agricultural area; and it is still more difficult to realize how little of this pressure comes from the man who is supposed to come on to this new acreage, pioneer to a large extent, make a living, if possible, and then pay for the cost of the project.

Many people draw an analogy from the industrial field and point out how products in an industrial line are absorbed beyond any forecast which has been made. They seldom stop to think that as far as staple food products are concerned there is a definite limit beyond which it is impossible to go after people are well fed. The analogy does hold in part in the case of what a few years ago were considered food luxuries and, as the standards of living become higher, the power to absorb such luxuries will increase. As to these luxuries, it will undoubtedly be many years hence before anything like what might be termed a saturation point is reached.

The picture that Mr. Mead paints is accurate and important, but the point to be borne in mind is that this is by no means the whole picture. As he intimates, one kind of picture would be drawn by engineers, another by the settler, another by the economist, and probably another by the politician. The consideration of vital importance is that these pictures fit into the frame of the law as laid down by the Congress of the United States; if they do not, they cannot be regarded as a fair presentation. Many of the policies advocated by the enthusiastic friends of Federal reclamation are good in themselves but, as long as they have to be paid for by money which is supposed to be repaid, there should be a definite understanding that a business transaction is involved and that obligations to be undertaken are as binding between the Government and individuals as between private parties.

G. E. P. SMITH,[43] M. AM. SOC. C. E.—Any discussion of a National reclamation policy must involve the question of the legal basis of Federal activity in land reclamation. This is fundamental. On what basis does the U. S. Bureau of Reclamation function? On what basis does Congress authorize an occasional new project? Is it by virtue of National sovereignty; is it by sufferance of the States; or is it through contract or understanding between the sovereign Federal Government and the sovereign States?

Fundamentally, this Federal activity rests on an ephemeral and insecure basis. National irrigation projects in the United States have been so monumental in character, so spectacular, and so widely heralded, that many people (perhaps a majority of them) now take it for granted that land reclamation was written or implied in the list of powers delegated to the United States in the Constitution. On the contrary, all matters of such character were retained under control of the individual States. It is by urgent invitation of the people —virtually by invitation of a State—that the Federal Government "enters the picture" and executes a project like that of the Salt River Valley, and it is the bonus or subsidy that makes the Federal aid acceptable. Without the subsidy the States, or the districts, with few exceptions, would execute their own projects in due season.

---

[43] Prof. of Irrig. Eng., Univ. of Arizona, Tucson, Ariz.

The Federal Government has limited delegated powers. Because the States felt that they had delegated power too generously, the first ten amendments to the Constitution were adopted at an early date to check the newly created "monster". In all ages people have had occasion to fear the power of governments. A stirring address[44] by Lincoln Bush, Past-President, Am. Soc. C. E. has served to emphasize the ever-present possibility of mal-administration either for political purposes or for private gain. The larger the unit of government is, the more difficult it becomes for the people to retain control of it, and there is increasing evidence that the United States Government has reached the safe limit. President Coolidge has stated that "unless bureaucracy is constantly resisted, it breaks down representative government and overwhelms democracy". President Hoover stated that:

> "The greatest need at Washington is to decentralize the National Government and to pass back into the hands of the people the responsibilities which have been gradually taken from them."

The authority for the Reclamation Act of 1902 is found in Section 3 of Article IV of the Constitution, giving Congress the power to dispose of and to make all needful rules and regulations respecting the territory and other property of the United States. This, however, is a proprietary right and not a sovereign power, and has been so stated, emphatically, by the Supreme Court of the United States.

Section 8 of the Reclamation Act provides that the Secretary of the Interior, in administering the Act, shall proceed in accordance with the laws of the States. If this provision had not been included, it would still be true that the Act could be administered only with the consent of the States. Therefore, at the outset of each Federal project, the Secretary of the Interior, or the Bureau of Reclamation, has made application to the State for an appropriation of water, in the same manner as an individual or a corporation.

The speaker is aware that many people, notably certain people of Southern California, would like to have the situation reversed, and some have come to believe that Federal sovereignty includes control of irrigation water supplies and of water power; but, no doubt, "the wish has been the father to the thought". A very few attorneys, who have been reared in the atmosphere of Federal bureaus and whose vision, therefore, is limited, have devised a theory of law by which they claim Federal ownership of Western waters. Those gentlemen arrived eighty years too late. Perhaps in 1850 the Supreme Court would have given consideration to their theory, but, to-day, when such theory would upset the machinery of law for control of water rights in the various States, and likewise require the reversal of a score of important decisions, the Supreme Court would give it scant attention. In a few cases in which the contention has been advanced, the Court has charitably ignored it.

---

[44] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 1646.

The general control over the waters of the State is a sovereign power and is vested in the State. The powers of the Federal Government are limited to the control of navigation. As owner and proprietor of lands that are irrigable from streams, the Federal Government has the usual circumscribed rights to utilization of the streams for the irrigation of those lands, subject to the sovereign control by the State. Many Supreme Court decisions can be cited in support of this, and the decisions are unequivocal.

The worst thing, therefore, that could have happened to the Imperial Valley was for the Swing-Johnson Bill to become a law as written. In that bill, Congress attempts: (*a*) To allocate a portion of the waters of the Colorado River between States; (*b*) to restrict the effective appropriation of water in the United States to about 80% of the water supply; (*c*) to develop 450 000 firm h. p. of Arizona's potential water power over the protest of the State, the benefits resulting therefrom to go to other States; and (*d*) to build a project of unparalleled dimensions, 88% of the cost of which can be justified only as a power project. The remaining 20% of the water supply would be allowed to flow into Mexico, where squatters' rights to it would be secured by shrewd and powerful owners of Mexican lands, and, later, these rights may be confirmed by treaty.

The Engineering Profession has failed in its inability to develop, and agree on, a logical, just solution of the whole problem while there was opportunity. The task belonged rightfully to the State engineers of the seven States, to evolve a plan of development that would be just and advantageous to all the States. The case will be precipitated into the Courts and may drag along through many years (as in other cases of water litigation) and in the end the Federal sovereignty will be defeated.

Imperial Valley should not be asked to wait twenty years for a reasonable degree of flood protection, nor to wait at all for an additional water supply for late summer, which admittedly is its greatest need. Both these desirable results could be brought to fruition in three years, if politicians and city boosters could be eliminated and well-established engineering principles could be brought to bear. The silt problem is a minor issue and will be solved incidentally.

Inasmuch as the proposition has been here advanced to differentiate between interstate and intrastate rivers, it becomes pertinent to examine the legal possibilities of this proposal. It is advanced as an idealistic solution. One practical effect would be to give California control over its water supplies, while the control of all water resources—main streams and the smallest tributaries—in States such as Idaho, Montana, Colorado, New Mexico, and Arizona, would be surrendered to the Federal Government. Presumably, this would have to be accomplished by an amendment to the Federal Constitution.

About 1916 the Committee of the Judiciary of the United States Senate was directed to report on the rights of the Federal Government relative to water power of streams within the respective States, and the report was made by a sub-committee[45] consisting of the best constitutional lawyers in the Senate. The majority report was written by Republican members, who might be expected to lean as far as possible toward Federalization. The minority report was even more pronounced in the declaration of State control of streams.

In effect, this report states that, as regards water power and irrigation, there can be no differentiation between interstate streams and intrastate streams, and in all matters other than those of navigation, the ultimate authority and control over waters rests in the State. The status of the Federal Government as owner of abutting lands is not different from that of other owners.

The Sub-Committee states[45] its opinion regarding the rights of the Government under the commerce clause of the Constitution, as follows:

"The mere grant by the Federal Government of authority to construct a dam in a navigable river, not for purposes of navigation, but really for the creation of a water power, is merely a license or permit, the effect of which is that if the dam is constructed and operated conformably to plans approved by the Government, it will not be deemed an obstruction or impediment to navigation. And in such case the Government would be authorized to charge a nominal license fee for inspecting and passing upon the plans and for watching over the work to see that it conforms to the plans and is properly maintained; but the regulative power of the Government would not extend to the use of the water for other purposes than navigation and interstate commerce. In such a case it seems to us that the Federal Government has no water power to sell or charge compensation for, for it is only authorized by the Constitution to regulate interstate and foreign commerce, which in this case means navigation."

Concerning non-navigable streams, the report contains the statement[46] that:

"The leasing of the water-power site as a part of the riparian lands of the Federal Government does not in and of itself confer the right to create a water power. At most * * * it merely confers the common-law right of usufruct in the water. If any other or further use of the water in the stream is required, the right to such use must be obtained from State authority, and, therefore, it is difficult to see how water power can be established in such cases without the co-operation or consent of the State."

The report of the Sub-Committee is based on the text of the Constitution and upon the interpretation of it by the Courts in many cases cited. There is no "pussyfooting"; the report is very conclusive. It should be mentioned, however, that in States in which riparian rights do not and never have existed, the case for the Federal Government is even weaker than that stated.

The object of the foregoing discussion is to emphasize two facts: First, regardless of what any individual thinks, the National policy relative to reclamation should be, that the States, and not the Federal Government, have sovereign powers over the control and utilization of streams in matters affecting irrigation, water power, and domestic use, and in all matters except those relating to navigation; and, second, that it is not desirable to change the present order.

---

[45] Senate Doc. No. 246, 64th Cong., 1st Sess.

[46] Hearings before Committee on Irrigation and Reclamation, U. S. Senate, 70th Cong., 1st Sess., on S. 728, January, 1928, p. 131.

If, however, it is deemed advisable and expedient that the Committee on A National Reclamation Policy should recommend an amendment to the Constitution delegating greater authority to the National Government with respect to control and utilization of the water supplies of the country, it is well to consider some of the obstacles that will be encountered.

First, consider the Pittman Resolution[46] adopted by the Conference of Governors and Commissioners of the Colorado Basin States at Denver, Colo., August 29, 1927. It is a carefully prepared, succinct statement and declaration of State sovereignty over water supplies. This resolution was developed to final form by the legal representatives of the Basin States and was signed by representatives of six States, the seventh, California, asking to be excused from voting.

Can the Committee cause a "right-about-face" on the fundamental attitude of the Western States as expressed in that resolution? Will those States be impressed by arguments relating to greater efficiency of control emanating from swivel-chairs 3 000 miles away? In passing, it may be remarked that every principle enunciated in the Pittman resolution is violated in the Swing-Johnson Bill.

It is not in the West only that the Committee will meet obstacles. New York State has engaged the Federal Power Commission in a death struggle over the control of water power of navigable streams (one of which is international), and the Federal Power Commission has gradually receded from its early assumption of jurisdiction. Will New York capitulate when its battle is just won? The State of Maine not only exercises control over its water resources, but has a law forbidding the exportation of hydro-electric power and thus far that unique law has held its ground. The Southern States also are locked in a struggle with the Federal Government over control of water power. Will they listen to an appeal from the Society to surrender their authority to the Federal Government?

The answer is, No. If the Committee advocates nationalization of the rivers, in any degree whatsoever, it will succeed merely in invoking violent criticism.

It is said sometimes that public necessity creates law; this is illustrated by the growth in power and influence of the Interstate Commerce Commission during the past two decades. However, in the matter of control and utilization of streams, the States are managing those functions satisfactorily at present. If one State feels aggrieved against another, it is not without legal remedy, for each State has the protection of the U. S. Supreme Court, and there is the alternative method of settling conflicts in interests by interstate compacts. Necessity cannot rightfully be urged as the argument for National control of the utilization of water resources.

If the Federal Government can say that a dam shall be built here and that there shall be no dam there, it involves the decision that one valley shall be granted rights of appropriation and another shall not; in other words, the States would lose control over appropriation of water.

It is probable that the compact method should be encouraged, at least in many cases. However, one of the most highly respected engineers, an authority on water law and a Californian, has stated his misgivings over the injection of the compact plan. Several excellent compacts have been negotiated, notably those between the States of Colorado and Nebraska and Colorado and Kansas; the latter, however, is not yet ratified. The Colorado River Compact was a "dud" because it was negotiated in great haste, for Congress in its unwisdom had decreed that the Commission should expire at the end of the year, and because, although some of the seven States had made elaborate studies of their future needs for water, others had made no studies and had no basis on which to negotiate. The Colorado River Compact could now be amended or rewritten so as to be acceptable to all seven States, if the commission could be re-established. President Hoover has suggested[47] that the Gila River System should be exempt from obligation to furnish water to supply the quantity to be allocated to Mexico by treaty. This amendment would be welcomed by the State of Arizona, if the other States should accept it.

If the functions of storage may be analyzed as three in number—river regulation, irrigation, and power—then it is desirable for Federal activity to be restricted in a large measure to regulation, to a less extent, to irrigation, and least of all, to power. Applying this to the Colorado River, the Federal Government should build storage for about 6 000 000 acre-ft. capacity on the head-waters. The expenditures required would be less than $20 000 000, and thereafter Federal aid would not be necessary for the combined irrigation and power projects which would inevitably follow at other points on the river. The Gila River is being brought under control by many dams, built and projected, and long records of stream flow prove that no other tributary between the mouth of the Green and the Gila Rivers is a menace to the lands along the lower river course.

Is it desirable to force rapid extension of reclamation? There are two distinct points of view prevalent with respect to this question. For convenience of reference they may be called the conservative and the promoter viewpoints.

The conservative viewpoint may be described as follows. If social and economic conditions warrant additional agricultural production, then men trained to the soil, who are ambitious to develop and own a farm, will appear voluntarily, or they can be found. These men, by early rising and sustained industry, by thrift and careful management, will transform a sagebrush desert into fruitful fields, as their fathers have done before them. This has been the rule in Arizona. The only deviation from the rule that the speaker recalls, was the building of many small homes on one corporation-owned project; but the people who moved into those homes soon went away.

A healthy, voluntary land settlement fluctuates with agricultural prosperity. When agriculture is prosperous, there is a strong demand for farms. In 1919, a project in Southern Arizona, with about 300 land units, was settled so rapidly—by trained farmers—that the land was literally taken away from the promoter. At the outset the price was $150 per acre for raw land with water, but when men stood in a long line waiting impatiently for their turn to be shown over the tracts, the price was raised to $250, with a cash payment of $75 per acre. As might be expected under the circumstances, the promoter over-estimated his water supply and over-sold the project.

---

[47] Conference at Grand Canyon, Ariz., summer of 1928.

The other viewpoint is quite different. It starts with the fact that potentially productive land lies idle. To reclaim it means interesting applications of engineering, local prosperity during the construction period, and the growth of near-by cities and villages. "Manless land" and "landless man" must be brought together, whatever the obstacles. If, as in the seven years since 1921 no land hunger exists, it must be stimulated; homes must be erected, fences built, the land graded and prepared for cropping, and some live stock must be placed in the corral.

There is a danger that, if much paternalistic effort is made for the settler, he will sit on the doorstep during the hot summer days, trusting that this mysterious Government which has done so much will come forward and cut the weeds and spray the bugs. Furthermore, such a beneficent Government certainly will not insist that he forego a new shiny automobile or a summer vacation in order to pay annual charges for operation and maintenance.

The psychological effect upon the settler in the two cases is entirely different. In one case, he realizes that his success depends upon his own initiative, his industry, and good management; in the other case, he starts with a feeling of dependency and is likely to adopt a "Micawber" attitude.

Paternalism in Government is based on an assumption of a *quasi*-fatherly relation, involving supervision of business and social concerns, on the theory that the people are incapable of managing their own affairs. It is ill-adapted to the virile Anglo-Saxon race. Why should the Federal Government build homes for farmers any more than for laborers where some other new industry is under way?

There is little doubt that big, costly projects have been developed too fast in the 25 years since 1903. If a 10-year holiday should be declared in reclamation, it is doubtful whether settlement would catch up with the supply of good land now available with irrigation water supply. If farming were merely a mode of living and land values were low, there would be some reason for arguing a "back-to-the-land" movement, in order to reduce the congestion of population and to increase the percentage of those who love the open air and the freedom of country life. Irrigation farming, under projects involving $80 to $160 per acre for water supply, is, however, a business proposition of the first rank and should be assured of rapid settlement and of success from an economic standpoint before any commitments are made. Inadequate settlement has caused more failures of irrigation projects than any other one thing.

In the case of large, difficult projects, governmental aid is necessary and justifiable, and the present form of this aid—the waiving of interest—should be called by its true name, which is subsidy. It is an important advantage to the country as a whole to have a successful agricultural project developed and put on its feet. If it is not successful, the advantage disappears; it becomes a liability.

To what extent is the subsidy justifiable? Surely the Federal Government should not create farms and deliver them gratuitously to a limited few farmers. The question of what percentage of the cost should be borne by the Government is too often neglected. It should be recognized that the major advantage goes to the farmer, the next in order to the community, the next to the State, and the last to the Nation. Why not adopt the plan used in financing elimination of grade crossings, by which the railroad, the State, and the city divide the cost. If a State is appreciative of the value of a new development and desires it earnestly, it should participate in the subsidy, instead of buying it from Congress through the demoralizing political method of trading votes. Many of the Federal projects have been, and are being, secured by purely "pork-barrel" methods. It may be advanced that the State is too poor; this is not true. If the State is unwilling to match the Federal aid, it is evidence that the project is unsound and ought not to be built.

Recently authorized projects of the Bureau of Reclamation have been over-subsidized. As an illustration, the Kittitas Project, in Washington, is designed to cost $160 per acre. The land owners are obligated to pay annually a percentage of the gross crop returns, estimated to average $2 per year,[48] so that the period of repayment may extend over 80 years. The present worth of a promise to pay $2 per year for 80 years is $39. Owing to the long interest-free interval before repayments begin, the face value of the paper contract is even less than $39. The farmer, therefore, is asked to pay less than one-fourth of the construction cost. Surely no one will claim that it is worth $121 or $140 per acre to the Nation as a whole to have that tract irrigated. It may be at some future time when the food supply is inadequate and when other areas are fully occupied, but not at present. It will be interesting, also, to see whether the land owners find some adroit way of capitalizing so great a bonus when resales are made.

There has been considerable protest, especially in the Middle West, against artificial Governmental stimulation of agriculture through extension of area. There is much more justification for protest by older, half-settled projects, especially those promoted without governmental subsidy, because the new projects compete actively for the limited number of competent potential settlers.

The U. S. Bureau of Reclamation has made an enviable record for the character of its works, which have been designed judiciously, with a view to appearance, safety, adequacy, and, in particular, permanency. Comparatively few engineering mistakes have been made. The thoroughness with which the relatively poor foundations of the Elephant Butte Dam were strengthened and protected reflects great credit upon engineers of the Bureau and the admirable *esprit de corps* infused in the organization by its first Director, F. H. Newell, M. Am. Soc. C. E., is a model for all time.

R. K. TIFFANY,[49] M. AM. SOC. C. E. (by letter).—The discussion of a National reclamation policy takes on new interest since President Hoover has proposed certain radical changes in the handling of Federal reclamation and in the closely allied subject of public lands in the Western States.

---

[48] *New Reclamation Era*, February, 1927, p. 18.

[49] Cons. Engr. (Tiffany & Langloe), Olympia, Wash.

The following suggestions may help further to clarify some of the ideas mentioned only briefly in the report:

1.—What is the fundamental basis of Federal aid for reclamation? Is it not to make habitable and productive for the public benefit areas of land that would otherwise be permanently desert waste? The Federal Government has taken up no projects that could be developed by other agencies.

2.—The steady growth of America has justified the traditional National policy of encouraging agricultural development. Recognizing the errors and occasional economic waste involved, who will deny the tremendous value to the whole Nation of the Homestead and Desert Land Laws, the Western Railway Subsidies, and the Reclamation Act?

3.—The desert lands of the West, although now largely in private ownership, are essentially a public burden. They cannot be developed by private enterprise, nor, in most cases, by the States.

The whole country needs the timber, mineral, wool, and other essentially Western products, and the growing commerce of the Pacific. To grow and prosper, the West must have, in due time, the production of its irrigated lands. The entire Nation will benefit from a policy that, under an orderly and economic plan, will make these lands available.

4.—The reference to "interest" in Conclusion (1) of the report should be understood to refer only to the period after the settler is upon the land, bearing in mind that the settler is doing a public service in developing the land, if there is a real economic need for its products.

5.—The Committee has outlined a National reclamation policy. It may be helpful in considering this question to have a tentative program for the immediate future. The writer suggests the following:

Tentative Five-Year National Reclamation Program.—

(*A*) Concentrate on study of means and methods to bring existing projects to successful conclusion, and turn them over to local management as rapidly as possible.

(*B*) Reconsider the so-called "Ten Year Program of Construction". Make a market survey of each district or project, covering local, National, and State markets for the probable production of each project and so modify the construction program as to bring in new land only when and where it appears that its products can be absorbed at prices profitable to the farmers.

(*C*) Apply the reclamation organization and any surplus funds to the building of reservoirs useful to flood control, irrigation, and power. These reservoirs and controlling works should remain under Federal control and should not be charged to the land.

(*D*) Study methods of decentralization by which States, or local organizations within the State, would build canals to be financed by the Federal Government under plans perhaps somewhat similar to Federal Land Bank organization.

DAVID WEEKS,[50] ESQ. (by letter).—This report is constructive in that it suggests a basis for broadening the scope of the activities of the Federal Government for the benefit of a larger number of American people throughout the entire United States. It is also to be commended because it provides for the apportionment of costs on a more rational basis than has heretofore been possible.

The Committee proposes: (1) Public control of water conservation and stream regulation; (2) allocation of costs among benefited interests, including the State and Federal Government; (3) disposal of water and power at wholesale; (4) regulation of interstate streams under interstate compacts; (5) construction at the present time for stream regulation and to provide supplementary water supplies for existing systems and not for bringing new areas under cultivation; (6) thorough investigation as to feasibility prior to construction by a board of review composed of engineers, agriculturalists, financiers, and economists; (7) repayment of principal, and a low rate of interest by land owners for costs assessed against farm lands; (8) control of land speculation; and (9) the relinquishment of control of completed works to suitably organized local agencies.

It is fitting that such a board of engineers should confine its findings to engineering matters. This it has done with the exception of casual recognition of the agricultural situation and its relation to reclamation and a brief reference to the important problem of agricultural development.

Federal Reclamation and the Agricultural Surplus.—Federal reclamation projects have provided homes for a rural population of nearly 150 000, and upon these lands crops valued at nearly $1 500 000 000 have been produced. Growing urban centers have been made possible in different parts of the arid West where industries vital to the development of other natural resources have been established. In these Western cities are large numbers of workmen whose families are supported by the agriculture of Federal reclamation supplemented by the products of range, forest, and mine. Automobiles, farm machinery, furniture, and other commodities are brought into these thriving communities each year from Eastern factories. A growing Western demand for Eastern products gives employment to those seeking refuge from the depressed rural areas and increases the purchasing power for farm commodities in Eastern urban centers. The taxable wealth of the Nation undoubtedly has been increased by Government activity in reclamation.

Nevertheless, the Eastern economist has asked, "What has been the cost of these benefits?", and Federal reclamation has been called upon to account for its share in the agricultural surplus. Those interested in the development of the Western States believe that Eastern economists have been unmindful of the real problems of irrigation development, and have diverted their energies from some of the more important issues confronting Western agriculture. Instead of expressing a desire to help to make irrigation a success, their one objective has been to prove that reclamation is a failure and that it contributes to the agricultural surplus. If it had not been for a group of Western enthusiasts who had desires equally as strong to establish a set of opposing facts, the statements concerning reclamation contained in the numerous lists of remedies for the agricultural situation, might have been taken as final. If the advocates of Federal reclamation have been a little too zealous in their determination to win their case, they have at least shown that there are two sides to the question.

---

[50] Prof., Coll. of Agriculture, Div. of Agricultural Economics, Univ. of California, Berkeley, Calif.

In 1927, less than 0.85% of the gross value of all crops in the United States was produced on Federal irrigation projects. Although small in comparison with the total value of all crops, commodities produced in these projects were of vital importance to the States in which they were raised.

Arguments have been written to the effect that although small in amount, the production on Federal projects has an important depressing effect on prices. These arguments are based upon the economic concept of inelasticity of demand. This is simply a technical way of saying that a small increase in production causes a large reduction in price. The limits of this discussion will not permit an elaboration of the many commodities produced on Federal reclamation projects. It might be well to consider the effect of Federal reclamation on the price of potatoes. In a similar analysis of this commodity the error was made of considering the effect of supply on price without giving recognition to the effect of price on supply. Consideration of the normal growth in demand was also omitted.[51]

The production of potatoes in the United States in 1928 has been estimated by the U. S. Department of Agriculture at 462 943 000 bushels. Since this production was more than 50 000 000 bushels above normal, the price was depressed. This excess above normal production was caused by conditions favorable to crop growth and to an increased acreage following the exceptionally good prices in 1925 and 1926. The activity of the Federal Bureau of Reclamation cannot justly be charged with these mal-adjustments inherent in the potato industry. The normal production of potatoes in the United States has increased since 1910 at the rate of about 3 500 000 bushels per year. Fluctuations above and below the normal trend have been accompanied by opposite tendencies in the price per bushel. (See Fig. 3.[52]) The normal increase in production on Federal reclamation projects has been about 500 000 bushels per year. (See Fig. 4.) The normal per capita production of potatoes in the United States decreased from 3.8 bushels in 1910 to 3.6 in 1920 and 3.4 in 1928. The trend of the purchasing power of potatoes has been upward for many years. The difficulties of the potato industry are caused by violent fluctuations in production and prices and not by an increase in the total normal supply.

The Net Effect of Federal Reclamation on the Potato Industry.—To say that Federal reclamation had no effect upon the potato industry would be as erroneous as to say it depressed prices 4 cts. per lb. Local cases could probably be found in which a shipment from some Reclamation Project might have upset some local market. The most important effect, involving a larger number of producers, would be that a certain acreage of land which, with the growth of the potato industry, would have been used for growing potatoes, has necessarily been devoted to something else. Without following out the intricate adjustments which might result from such alternate use of land, the net result might logically be the substitution of a comparatively small acreage which is producing potatoes in Reclamation Projects for a somewhat larger area of stump land in the East or West which otherwise might have been cleared for producing potatoes or some other crop.

---

[51] See Millard, Peck, "Reclamation Projects and Their Relation to Agricultural Depression: Farm Relief," *Annals*, The Am. Academy of Political and Social Science, Vol. 142, 177–185, Philadelphia, Pa., March, 1929.

[52] Year Book, U. S. Dept. of Agriculture, 1928, p. 806, Table 202 (1929).

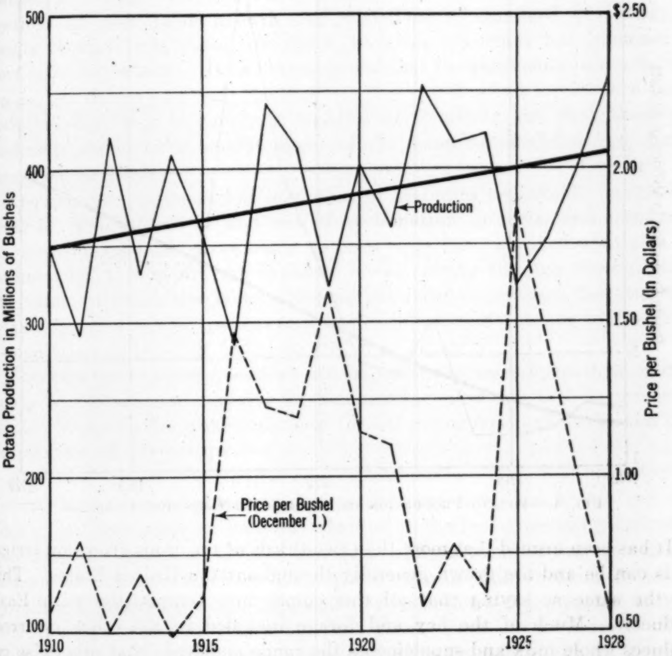


FIG. 3.—TREND OF POTATO PRODUCTION AND POTATO PRICES IN THE UNITED STATES.

If production in Federal projects had been suddenly superimposed upon United States production, especially with respect to potatoes, economic forces would have set about reducing production in succeeding years. This would have taken place on a few farms, marginal for potato production, in and out of Federal projects. The result, over a period of years, would have been a normal production and the price but slightly different from what would have existed without Federal reclamation.

Instead of being superimposed upon the production of the country as a whole, however, the acreage of all crops added each year since 1913 by Federal reclamation has been only about 76 000 and of potatoes the average acreage added each year has been less than 2 300. Even the addition of this small acreage, if not offset by increased demand, would be followed by adjustments in production if the effect upon price was such as to warrant it. In the meantime, however, there has been some increase in demand. Indeed the annual fluctuations in production from other causes probably have so far overshadowed the effect of irrigated potatoes that statistical analysis would fail to find it.

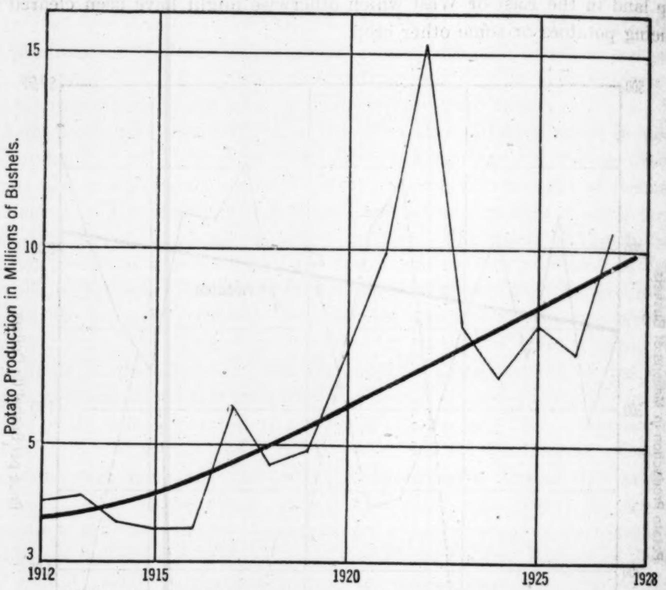


FIG. 4.—POTATO PRODUCTION ON FEDERAL RECLAMATION PROJECTS.

It has been argued that more than two-thirds of the crops grown on irrigated lands can be and are grown generally throughout the United States. This is not the same as saying that all this comes into competition with Eastern producers. Much of the hay and forage included in the crops referred to produces whole milk and supplements the range and feeds that otherwise could not be fully utilized. The remainder, if it finds its way into Eastern markets in one form or another, either supplements other sources of supply by filling in seasonal deficiencies and reduces imports, or fills a demand for a special variety or quality of product.

The situation is somewhat different in California where perennials are the most important irrigated crops, but these difficulties again are inherent in the particular crop industries of that State. Difficulties in adjusting land utilization to market demand and the tardiness with which decreased production responds to decreased demand result from the years required for a tree crop or vineyard to come into full bearing. Distress resulting from over-production of individual crops because of the market and production characteristics of these industries should not necessarily be charged to reclamation. It is, however, one of the difficulties which reclamation development faces and no doubt reclamation in certain local areas has forced land into the production of crops already in a condition of over-production.

Does Reclamation Create New Life and New Wealth?—An outstanding omission in the arguments against reclamation is the failure to recognize the organic growth of Western communities made possible by irrigation development. The many arguments concerning the extent to which reclamation creates its own demand have been founded for the most part on inadequate knowledge. It is a difficult question which cannot be answered by mere compilations of figures on imports and exports. There are the fundamental questions of population growth and distribution, standard of living, and psychic returns from rural life which have an important but immeasurable effect upon net benefits. It has been argued that the shipments into a reclamation project do not represent the creation of a new demand for Eastern factory products; that this is simply a transfer of demand; and that these same purchasers would have bought many of the same commodities had they remained in the East.

The counter argument has been that of statistics on taxable wealth, shipments of commodities in and out of reclamation projects, and numbers of churches and schools. Arguments on both sides have wreaked with bias. No one has tried to measure the dynamic forces arising through the building of new communities in the West. No one has tried to measure the extent that reclamation creates new life and new wealth in contradistinction to the transfer of population and wealth from one region to another.

The answers to these questions are important because upon them rests the answer to Government activity in reclamation—how far the Government is justified in giving financial assistance for the construction of works and in the development of agriculture.

Economic Questions Confronting Reclamation.—The real question confronting irrigation development during the next few years is not one of its effect upon Eastern agriculture, but whether, in the face of existing agricultural conditions, an irrigation project can gain sufficient momentum to come into production with construction costs still high and with incomes from many agricultural enterprises still lagging. Urbanization of population is taking place at a rate which is increasing each year. That the percentage of rural population is declining is only a symptom of two important facts in relation to reclamation. In the first place, it indicates that cities are offering better opportunities for the masses than the farms and, in the second place, it is an indication of important changes in methods of agricultural production.

Agricultural production is becoming industrialized. Capital is replacing man power. How fast and in what form this tendency will take place in the future is not known but it is becoming increasingly difficult to create a farm under irrigation on a par with those already established. Until more adequate machinery is set up for financing modern agricultural development in a project after it is brought into existence, it is doubtful whether new reclamation, especially under present conditions, is possible.

Inasmuch as such financing of agriculture has not met with favor and because the possibility of doing so has not been successfully demonstrated in the United States, it may be logically concluded that reclamation of new areas at the present time is likely to result in the loss of a large part of the funds thus invested. A policy of curtailment is, therefore, wise. Two important observations should be made, however. Curtailment of reclamation, except in certain local areas, as a solution of the agricultural surplus problem is of minor significance. In fact, on the basis of knowledge now available, it is safe to state that the promotion of reclamation is just as likely to help the agricultural situation as to impair it. This question, however, is in need of more intensive study. Of greater significance than its effect upon the agricultural surplus is the reclamation policy, which should evolve from a desire to help reclamation to success rather than to bend every effort toward its destruction.

The plan proposed by the Committee, unless supplemented by a more adequate consideration of agricultural development, thus perpetuates the onesidedness of reclamation. Irrigation works have usually been readily financed. The plan of the Committee will make it still easier to bring large irrigation works into existence. A lag of agricultural development behind irrigation construction is one of the chronic ills of reclamation. It is not peculiar to periods of depression. It represents one of the most important costs. The lag between irrigation construction and agricultural development will be increased if a supplementary plan is not worked out, which will make possible the synchronization of agricultural development and irrigation construction. To synchronize agricultural development and the construction of irrigation works, it will be necessary to build projects more in accordance with agricultural needs and then to bring them promptly into production. If the Federal Government engages in reclamation, specializing in the construction of storage works, agricultural development should be the "concern" of the Federal Government at least to the extent that contracts with States, for the joint financing of works, should carry provisions stating how that important phase of reclamation is to be done.

Economics of Agricultural Development.—Perhaps wisely, the Committee avoided the difficult agricultural and economic problems of land development under irrigation. It is because they are difficult and not well understood and because they are not engineering problems that they have not been given more consideration.

"In the case of reclamation projects, it should be recognized that settlement of the land is fully as imperative to success as construction." This statement the Committee follows with the further assertion that,

> "Land settlement including paternalistic, financial, or any other kind of assistance to the individual farmer should be treated as a local matter and should therefore be made the concern of the State or locality rather than of the United States. Aid extended by Federal Land Banks has been generally helpful and the possibility of further extension of such aid by this or similar agencies is worthy of serious consideration."

Recent experiments in land settlement and rural development in California have met with unfortunate circumstances, which have been given various interpretations by the public. No attempt has yet been made to give proper weight to the different elements which have caused the difficulties. The general conclusions have been that the plan failed. Therefore "the entire plan is wrong". History is replete with similar instances. A close investigation of the facts of the case would reveal that the California experiments were overtaken by the same agricultural depression which has kept Congress busy since about 1920, that construction was carried out at high costs during a period of optimism and repayment and settlement took place under conditions of low agricultural prices and pessimism. There were many other elements in the situation, however, some of which might rightfully be charged to fundamental weaknesses in the original plan, others which were injected into the plan by ambitious legislators and public officials, and still others arising from the newness of the undertaking. No attempt has yet been made to profit by this experience and to perfect the plan or even to make an entirely new plan.

This attitude toward land settlement is in part caused by the recognition that there is plenty of "dynamite" in any plan of land settlement and in part to the attitude of the public toward agricultural expansion. Both these reasons have prevented the further development of land settlement methods and they account for the attitude of this Engineering Committee in proposing to have its monumental structures built as far as possible away from this "dynamite". Those who have fearlessly recognized the importance of this aspect of reclamation to the extent that they have thrown their entire lives into it, have been criticized by a public and an Engineering Profession which have not been educated to the importance of irrigation development in the West and which have not understood the real need and purpose of working out a technique of agricultural development.

Needed Institutions for Agricultural Development.—Outside the California experiments and inadequately supported efforts on the part of the Bureau of Reclamation, no attempt has been made to devise some form of institution for local improvement and government which will give to rural areas the same advantages that municipal governments have in constructing and financing development much broader in scope that the mere construction of irrigation works. When such an institution is created, it must meet the rigid requirements which the American public demands.

There Must Be Provision for Financing Modern Farm Development.—Existing credit agencies shun new agricultural areas. This is because they have no assurance of having principal with interest repaid to them. Their experience is that defaults have been numerous. The settler has not had the opportunity either to demonstrate his own ability or the productivity on his farm. The hazard, therefore, is great. When commercial firms decline to make loans on the basis of 50% of a conservative appraisal, how can an institution be set up for extending even more liberal credit with assurance of having it returned? One answer to this question is, "Don't do it". Another answer is, "Reduce the hazard by selecting the settler, classify the land, require repayment on a basis that will make it possible for a man of small means to meet the installments, and give him all the advice and direction possible while he is improving his farm". The first leaves the problem unsolved. The second suffers the criticism of being paternalistic.

The Plan Shall not Be Paternalistic.—Webster defines parternalism as the:

"Principles or practices of a Government that undertakes to supply needs or regulate conduct of the governed in matters affecting them as individuals as well as in their relations to the State and to each other on the assumption that it can best determine and secure their highest welfare."

It is argued that the liberal extension of credit and the giving of advice and direction in the early stages of farm improvement is paternalistic to a degree that settlers acquire the habit of leaning on the Government in matters in which they themselves should learn to become self-dependent.

Reclamation and Agricultural Development Must Be Free from Political Influence.—Hand in hand with paternalism is political interference with economic tendencies and practical business necessities. Political influence has caused infeasible reclamation projects to come into existence and has obtained alms for the unhappy settlers who were induced to locate therein. State and Federal Government agencies charged with the execution of reclamation and agricultural development plans should be as free from such influences as it is possible to make them.

Cost of Reclamation and Agricultural Development Should Be Apportioned Among Those Receiving Both General and Special Benefits.—If benefits to the Nation as a whole could be determined accurately and if, when determined, political machinery and the Courts would permit assessment of the Federal Government for those benefits actually enjoyed by the American people at large, the question of a policy of subsidizing reclamation would be a simple one, complicated, however, by questions regarding the proprietorship of Western oil and mineral resources. Benefits will probably continue to be determined inaccurately. Courts will adhere to precedent in regard to assessments for general benefits. Political machinery will probably continue to be arbitrary in the formulation of public policy. Ownership of the resources of the Public Domain will continue to be the subject of fiery debates in Congress. Subsidy, under such circumstances, cannot be granted on a basis commensurate with public benefit. Moreover, even with subsidy, many of the ills of reclamation would remain unsolved. Dams could be constructed and irrigation canals built at no cost whatever to the settler, and there would still be difficulties to overcome, greater than any of those surmounted by the engineers in the construction of such dams. The question, therefore, of deciding for or against the subsidy in reclamation is not the most important to be determined in the formulation of a National reclamation policy.

A Suggested Plan of Procedure.—The first step to decide is whether reclamation in the Western States is conducive to sufficient public benefit to warrant an attitude of support rather than antagonism on the part of the Federal and State Governments. Anticipating the outcome of such deliberations, it may be decided that it is best to curtail the reclamation of new areas for the present, but that it is wise and constructive policy to do what can be done to make existing projects successful.

The difficult question at once arises as to what crops can best be grown on the various lands, now within projects, which are lying idle or are producing crops that cannot pay the costs levied against them. In this connection the rate of assessment, availability of settlers, their probable qualifications and capital, types of farm organization, size of farm, methods of production, and total capital requirements must be considered, in addition to physical factors affecting crop adaptation and economic factors of prices and markets.

Having determined the optimum combinations for successful agriculture, the problem which is still unsolved must be faced. Where is the capital coming from? Should it be supplied by the Government? The last question has been answered by popular acclamation with a decisive, "No!". Whether this answer is right or wrong it represents a situation which is an element in the whole problem. The alternative is to make the financing of farm development safe enough to attract private capital. Before this can be accomplished experiment and demonstration will be necessary. If it is unwise for the Federal Government to engage in supplying credit for agricultural development as a regular practice, at least it would seem permissible for it to develop from ten to twenty farms on a purely experimental basis. This has long been recognized as being within the proper scope of governmental activity. These experiments should be carried on without limitations as to the total amount of capital to be applied to any one farm, as has been done in the past.

What is needed is more information regarding what that limit should be. Instead of cutting the farm to a size to fit the settler's pocketbook, the size should be determined by experiment at the optimum. Of course, ten or twenty farms would not answer all the questions which are still unanswered, but it would be a means of bringing out the important elements. Overhead expenses could not be charged to this small number of farms. Adequate capital should be appropriated for the experiment which should prove of value to agriculture under irrigation throughout the West. Part of this appropriation would be returned through the repayments on the farms. A part must be charged to experiment. The first experiment should be free from the difficulties of community organization. Later, with the knowledge gained, a special study could be made of the advantages and disadvantages of group settlement.

Because of differences of opinion on these matters held by the U. S. Bureaus of Reclamation and Agricultural Economics, the research and experiment should be directed by an economist selected by the joint action of both Bureaus. It should be the duty of this economist not only to develop technique of farm development under irrigation and determine capital requirements, but to suggest the type of administrative organization and financing program necessary to make possible the successful completion of reclamation projects.

JOHN A. WIDTSOE,[53] ESQ. (by letter).—The two supporting papers of this report contain views that will contribute much to the development of a successful reclamation policy. They do not mention, however, the most important fact connected with the Federal irrigation movement, namely, that land reclamation as a National activity, is rooted deeply in National policies, not politics. Federal reclamation is an expression of a twofold desire: To possess the entire country by full and profitable settlement and to establish more firmly the most important source of National stability—agriculture—by providing landless, land-loving men with farms. That is a National concern. National reclamation policies, therefore, must look to ultimate rather than to immediate results.

---

[53] Liverpool, England.

There is a veiled fear in the report of the Committee that Federal irrigation may interfere with private business, and that rigorous restrictions, therefore, must be set up. Such interference would be a misuse of the Reclamation Act. Federal reclamation activities should concern themselves primarily with projects that private enterprise cannot, or will not, undertake. In fact, this has been done. Few, if any, of the splendid major structures of the Bureau of Reclamation would have been built with private capital. The Government ventures have encouraged, not competed with, individual enterprises. The greatest recent private irrigation development in America has been inspired by, and coincident with, the growth of Federal irrigation. The wealth of experience gained on Federal projects, systematically gathered and published, is used profitably on private enterprises. Indeed, the Federal projects have not brought the Government into business, but rather have increased private business.

The present field for Federal irrigation is probably the completion and rounding out of existing projects, private and governmental. Many private projects are unfinished and they represent large money losses. The Reclamation Fund could bring these into profitable operation, and salvage some of the losses. Other projects, established and profitable, face with dread the inevitable dry year. Insurance can be provided by a supplementary supply of water from high-priced, storage reservoirs, which can be built on the terms of the Reclamation Act, but are beyond the scope of ordinary banking practice. Then, also, small pioneering communities, "in the very raw," are scattered about in the desert, waiting for help—aside from their own toil—to construct the little reservoir and canal necessary to bring the meager reward of their efforts. Such enterprises are disdained by the banker; for such communities the Bureau of Reclamation can render the service clearly implied in the original Act.

Reclamation is not accomplished overnight; it requires periods of time beyond those recognized in the market place. Financial elasticity must be available for successful reclamation. Men cannot, under ordinary conditions, reclaim the desert in one year or in ten years. Men grow old and leave their sons after them to convert the dry desert into desirable homes. Governmental assistance must be given often at an immediate loss, but with ultimate gain The form which this assistance took in the Reclamation Act was the use of interest money in the construction of the projects. The capital was to be repaid, but no more was expected of the water user. No better or fairer plan has yet been devised. The suggestion of the Committee that the necessary assistance should appear in "the assumption of a part of the cost of project works" and that the water user should pay interest on the remainder, seems fraught with much danger and difficulty. Primarily, it invites loss. How much of the cost should be assumed? Should the first settlers, when the project is new and struggling, pay the same rate of interest as those who come on later, when much of the pioneer work has been done? It would be better to cling to the wise statesmanship of the original Act.

Lack of knowledge concerning the principles involved in reclamation by irrigation has led to considerable losses on Federal projects. Much has been learned since 1900. The errors of the past need not and will not be repeated. Looking forward in irrigation is the present problem. The past can be used to help in the solution of difficulties and not as a means to frighten later settlers.

Among the mistakes of the Federal irrigation venture, none is more glaring than the assumption that to provide water for land is the essence of a successful irrigation project. The human factor is ignored. The error, indeed, went further in the assumption that water was more important than land or man. Alkaline water on good land, or good water on land of low productivity, spells failure; the best of water on the best of land will yield no success unless the man, by inborn power and by necessary help, can use the combination of water and land profitably. The stern dictum of the future must be: (1) Do not impound or divert alkali water for irrigation purposes until Science can show how such water may be used safely; (2) do not build irrigation works for lands of low productivity, until Science has taught owners how to farm them profitably; (3) supply the water user (the man who is to combine water and land into a unit of civilization) with knowledge, tools, and money, if necessary; and (4) vary the annual repayment with the possible annual income. If the Reclamation Service is foolish enough to bring water to infertile lands, or to lands beyond favorable markets, then it must be content to let the water user have ample time in which to pay.

Mr. Mead's contentions are sound. Successful irrigation cannot be expected in the future unless social and economic conditions are given first consideration. The best of engineering structures—ample water, fertile soil, and good markets—are demanded; but the man who uses all these factors, determines the success of the combination, and his needs, which are more difficult to meet than the stresses and strains of the dam, must be the concern of the new day in reclamation by irrigation.

One cannot have an intimate knowledge of conditions in Europe and take very seriously the cry of agricultural over-production through irrigation expansion. The relative area under the ditch is too small and the rate of increase in population is too great. With every coming decade men will look for more land, not for limitation of the agricultural area. Lands are reclaimed under the ditch, slowly, and the building of new irrigation works and the opening of new projects come with equal deliberation. It cannot be otherwise.

F. H. NEWELL,[54] M. AM. SOC. C. E. (by letter).—The value of a National reclamation policy must be judged by the practical, not theoretical, effect of such policy on the prosperity, the comfort, and the health of the Nation as a whole. It is to be assumed that the report of the Commitee of the Irrigation Division has this National, as contrasted with local, viewpoint in mind. If, for example, the drift of affairs should be such that the Federal policy, in promoting the reclamation of 1 000 000 acres, should result in the delay or should discourage the States, irrigation districts, or corporations organized under State laws, from reclaiming 2 000 000 acres of better land, then there must be something wrong. Not that this is an established fact; the tendency is there.

---

[54] Formerly Chf., U. S. Reclamation Service; now Pres., The Research Service, Inc., Washington, D. C.

The first point made in the report, that of the unwisdom of waiving interest payments, emphasizes this tendency. Its effect is that of a subsidy, one by which less than 7% of the owners of irrigated lands, those on Federal projects, enjoy interest-free advances, while more than 93%, that is, land owners on State or corporate projects, must pay in full. To state it in another way; "It appears unfair that the farmers on 392 000 000 acres in the United States should be taxed for the purpose of supplying a large subsidy to those on only 1 803 000 acres, supplied with water by the United States." If this condition exists then the answer seems obvious as to why irrigation is not more advanced in many States and irrigation districts, and also why efforts are concentrated on getting more Federal appropriations. As long as kindly Uncle Sam, with his boundless wealth, will advance the money, it would be a waste of time to bother with other ways and means.

This easy credit, however, has not been an unmixed blessing. A reason assigned for the large percentage of failures of the farmers on Federal projects has been that these men, stimulated by the prevailing speculative spirit, have been less cautious; they have tried to handle 80 acres or 160 acres and, with lack of capital and experience, they have fallen deeply into debt. They might have fed their families, got less into debt, and acquired skill on smaller units.

The way in which this remarkable situation has come about, should be kept clearly in mind in evaluating the points of the Committee report. Most engineers have forgotten, if they ever knew, how these remarkable precedents of easy credits were established; how it happened that land owners on Federal irrigation projects are exempt from interest payments and form a class apart from other debtors of the Government. To get the full import of the Committee report, it is desirable to sketch briefly the points from which a start was made and the direction taken during the last quarter of a century in the development of the present policy.

When the Reclamation Act was signed by President Roosevelt on June 17, 1902, it had been preceded by nearly fourteen years of investigation initiated by Maj. John W. Powell under the authority of Congress (Acts of March 20 and October 2, 1888). Topographic and hydrographic surveys leading to the selection of various desirable projects, by 1901, were well under way. Thus, when Theodore Roosevelt became President it was possible to give him sufficient information to enable him, in his first message, to bring the needs of the country forcibly to the attention of Congress.

During preceding decades rapid progress had been made in irrigation; millions of acres of arid land had been irrigated by individual or co-operative effort; other millions of acres had been selected or purchased for reclamation. The rate of growth of irrigation at the beginning of the Twentieth Century, however, was slowing down. The economic reasons for this were not then fully understood. Attention was concentrated upon certain obstacles which might be removed by the Federal Government and which, if removed, it was believed, might stimulate further development by individuals and State aid.

The great waves of westward movement of people, which marked the Nineteenth Century, had begun to subside, but the impulse still remained; there was demand for more land to be opened. This was on the assumption that the pioneer spirit still continued. Here was a fallacy; the true spirit of pioneering had been overshawdowed by the spirit of speculation. What really dominated was the thirst for pioneer profits rather than for homes on the land. Appeals had been made to each of the Western States to recognize its opportunities and responsibilities. As an entering wedge the Carey Act was passed in 1894 permitting the States to select public lands and initiate reclamation projects within their boundaries. Much enthusiasm was aroused, but the States did not rise to the opportunities; the conditions were such that under the Carey Act neither the real pioneer farmers nor the investors succeeded.

Without going deeply into the economic conditions, it was then assumed that a new impetus would be given to Western settlement if the Federal Government would take the lead in a series of experiments. Year after year unavailing efforts were made to obtain appropriations from Congress for building irrigation works. It was not accomplished until the rather ingenious scheme was devised to avoid making an appropriation. The result was achieved by diverting certain moneys before they became part of the general funds in the Treasury. This scheme circumvented the "watch-dogs of the Treasury" in the Committee on Appropriations.

It was urged that by the use of this special fund the Federal Government could build certain large works, the nature of which was such that they could not be undertaken by private capital, or even by the States, for the reasons either that the lands were owned by the Federal Government, or because there were certain interstate and international difficulties involved, or from the fact that the first costs were so large as to be practically prohibitive.

This condition should be kept in mind in order to understand the starting point of the National reclamation policy. In choosing among the large number of projects presented at that time, one of the first requirements was that any project to be considered should consist of public land; next that there should be some obstacle insurmountable by private agencies, such as State lines or of rights of a character such that they could be acquired only by the Federal Government. Moreover, it was a requisite that the enterprise should be of such nature that it was improbable that private capital could succeed.

At that time it was not a question of choosing the easier or more attractive projects; rather the reverse. Practically all the easy or cheap enterprises had already been undertaken; the various States had already selected, under the Carey Act, millions of acres including schemes which then seemed practicable. The requirement was that the Federal Government should undertake in each State one of the "rejects", or an enterprise so vast that private capital, even if aided by the State, could not hope for success; or it should confine its efforts to places where interstate and international complications existed. For example, one of the first determined upon was the Milk River Project, on the Canadian border, necessitating a treaty with Great Britain. Another was the Newlands Project, in Nevada, involving interstate relations in storing water and on which large areas of public land then remained.

It is possible to go down the list and indicate for each of the original enterprises the particular difficulty to be overcome and which formed the basis for selection. Some of these obstacles have been forgotten, but at the time they were the chief determining factors. In each case as careful a study as possible was made, not only of topography and hydrography, but also of climate, soil, prospective crops, and markets, as well as of the economic features which might be expected. There was, of course, relatively little known at that time concerning the behavior of soils under irrigation, but the experts of the Department of Agriculture were consulted. They subsequently conducted experiment stations on the projects such as to make available a vast accumulation of facts for future guidance.

This National reclamation policy was gradually modified in detail as successive Secretaries of the Interior were called upon to act in specific cases. The principal changes in the original policy came when Secretary Lane, confessedly weary of endless importunities, took steps to put the responsibility upon Congress. Here was the breaking point from what might have been called the Rooseveltian policy of executive decisions. Up to that time the various Secretaries of the Interior, co-operating with the Secretaries of Agriculture, had selected the best available experts, appointed committees or commissions, and had acted on the advice of these engineering, agricultural, or economic boards. After Secretary Lane set aside this policy there resulted, in the opinion of many engineers, a sacrifice of scientific and technical expert opinion.

According to the ideals of the founders of the National reclamation policy, the results should be judged, not so much by the area of lands irrigated, nor by the total value of products, as by the influence for good upon individuals and associations of citizens. Federal reclamation has added 10%, or less, to the area of irrigated lands in the arid region. It has done this by overcoming what at the time seemed to be insurmountable obstacles and not by following the easy course. In studying the results of reclaiming this relatively small area of irrigated land, by the use of Federal funds, the question may well be asked whether the outcome has helped or hindered further developments. Has it aided in the prosperity of the 90% of land irrigated by other than Federal funds? Is the Federal Government carrying out its early promise of stimulating individuals, communities, and States to increased activities in building up the arid West, or the reverse?

The nation-wide publicity given the affairs of the beneficiaries of the National bounty has tended to obscure or minimize in the minds of the public the far greater activities of the ordinary agencies of reclamation. The situation, to-day, in this respect is illustrated by the condition in Palestine where the well being of the 10% of Jews impresses the world as being far more important than that of the 90% of Arabs. So in the mind of the public the prosperity of the West has appeared to be tied up with the 10% of land owners under Federal projects; the great growth in agriculture, industry, and transportation through the 90% of private and corporate effort appears to be forgotten. To this extent results are not good.

In another respect also those acts, sometimes regarded as Federal interference with States' duties and opportunities, have the result of delaying various enterprises. It has been argued that the defects of the Carey Act would have been quickly remedied if hopes had not been pinned to the Federal Reclamation Act. This was passed at about the time that financiers and statesmen were discovering remedies for the difficulties encountered by State administration of irrigation enterprises. It is argued that instead of stimulating the States—as had been hoped when the Federal Government began to overcome physical obstacles, as well as interstate and international difficulties—the reverse took place; the State officials, instead of emulating these efforts, sat back to see the outcome.

Criticism has been made of the waiving of interest payments to be made by the owners of lands which have been reclaimed and made valuable by the use of Federal funds. At the time that these payments began to fall due, the idea was disseminated (and not discouraged) by Secretary Lane, that the rich and kindly Uncle Sam would not oppress any land owners; on the contrary he would grant them every possible favor. This attitude was in effect an invitation, especially when coupled with the statement, made on the floor of Congress at the time of the passage of the Reclamation Act, to the effect that the land owners would repudiate these debts. Thus, there came about an active campaign, not of outright repudiation, but for an indefinite extension of time. The effect of this was not confined solely to debts owed by land owners to the Government; it was seriously argued that other wealthy institutions, banks and loan companies, should also be generous. On certain Government projects the spirit of repudiation was so strong that financial houses hesitated or declined to do business in the localities, because of a general lowering of moral or financial standards due to the encouragement laid at the doors of Federal officials. It is not to be inferred that there was not real hardship in many cases. Under the present system, however, full discretion cannot be used by Bureau officers. All debtor land owners must be treated alike; hence, the breakdown.

Remember in this connection that the Secretary of the Interior deals with the people on the project, not as farmers, but as land owners; and also that a large proportion of these land owners (debtors of the Government) do not cultivate the land themselves; they are non-resident landlords. Some are bankers, business and professional men, clerks, and laborers trying to earn a living in town and renting the land to tenants. Due to these conditions the burden laid on the back of the actual cultivator of the soil often has not been diminished by executive leniency. Because of these easy terms given to the land owners and resulting speculation, the land prices have sharply advanced. The man who actually works in the fields has not been benefited.

The point which immediately concerns engineers, economists, and other citizens is as to whether the States, the irrigation districts, and corporations have been stimulated by the Federal activities. It may be asked whether these agencies might not have been more effective if the Federal Government had kept out of the field; the course of events might have forced the States to attack the reclamation problem through the terms of the Carey Act or some needed modification. All this is hypothetical, but leads to the conclusion that, with the experience of a quarter of a century, there should be a modification of Federal reclamation policy along the lines proposed by the Committee and one that will bring about a larger and better prosperity through encouraging these State agencies.

A. D. LEWIS,[55] ESQ. (by letter).—After a tour of inspection in America in 1913 the writer commented upon the irrigation situation as follows:[56]

"The result achieved in settlement is, after all, the measure of success in irrigation. Failures in irrigation schemes have resulted not so much from bad engineering as from lack of a proper understanding of the agricultural conditions and of all that is involved in the human aspect of settlement". * * *

"The chief lessons learnt were that settlers are not attracted to these schemes as rapidly as was expected and that difficulty is experienced in getting full payments from a large proportion of the settlers. * * * The supply of settlers with sufficient experience and capital is limited. * * * It is worthy of note that the American Government has not yet adopted the policy of loan advances (in cash or stock) to settlers to help them over the first few years."

During the sixteen years that have elapsed, the U. S. Reclamation Act has frequently been modified, numerous Commissions have reported, and the Reclamation Service has been unjustly abused and re-organized, but it appears that no complete solution of the difficulties noted in 1913 has yet been evolved in America. The history of Government efforts in irrigation has been very similar in South Africa, although conditions are less favorable to success. The surface of the country is very steep and denuded. The supply of water from annual rainfall and other sources is very small and erratic, and the cost of constructing storage works is excessive. The entire problem is further complicated by a heavy silt load in streams. The population density is less than 10% of that in the United States; marketing conditions are worse; and the tendency of political agencies to rush forward with new projects or to obtain special patronage for old ones is at least equal to that in America.

Despite these difficulties, irrigation received a tremendous impetus because of the ostrich feather industry, which is peculiar to South Africa. Dealers were prohibited from exporting the birds and this resulted in a very profitable monopoly of the market. Alfalfa was necessary for raising ostriches and it could be grown locally only under irrigation. From 1906 to 1913 the average price of feathers exceeded $13 per lb. and profits of at least $300 per acre yearly were made. As a result of these large profits, vineyards and other established crops were uprooted from the existing irrigated lands and were replaced by alfalfa crops. There was a frantic rush to build new irrigation works, many of which were merely primitive furrows to divert the intermittent flood flows from rivers. The industry was based on a fickle, feminine fashion and "all the eggs were in one basket." The standard of profits was far greater than the general average for irrigation farming. In 1914, fashions changed, the World War broke out, a severe drought occurred, and the industry crashed. The value of the export of feathers fell from $15 000 000 in 1913 to less than $20 000 in 1918, and a post-war agricultural depression supervened.

---

[55] Director of Irrig., Union of South Africa, Pretoria, Union of South Africa.

[56] "Irrigation and Settlement in America," by A. D. Lewis.

From 1914 there was a feverish demand for storage works generally to alleviate the unfavorable conditions and especially to save the farmers who had spent their capital on flood-diversion projects. To date (1929) the total expenditure for irrigation in South Africa is about $35 000 000 and nearly three-fourths of this expenditure was authorized by Parliament during the period from 1914 to 1920. It is now almost certain that the Government will not recover even half of this sum. Had the works been spread over a longer period and had active steps been taken to check speculation and to insure the rapid settlement of the area brought under irrigation, there might have been hope of recovering a larger part of the expenditure, but certainly not all of it. Since 1924 very few new projects have been sanctioned and the question of subsidizing future works is now under consideration.

Some of the projects are built and operated by the Government, but most of them are managed by co-operative irrigation district boards to which the Government has loaned money at 5%, payable over a period of thirty or forty years. Most of the land is privately owned, and the unit costs per acre of irrigable land have been about $130, which is probably double the cost of projects in the United States. In South Africa the physical difficulties are greater, but the higher prices that prevailed after the war (when most of this work was done) accounted for part of the higher costs. The better projects, however, were begun first and the unit cost of future developments in arid areas is likely to be even higher.

It will be seen, therefore, that difficulties in South Africa and in the United States are very similar. Engineers in South Africa have followed every phase of irrigation activities in America, always watching for the discovery of some magic process which, without any direct financial loss to the Government, will rapidly convert the desert places into smiling gardens peopled with healthy and prosperous families. Lack of success in the United States warrants the fear that there is no such process.

By waiving interest charges, irrigation projects of the U. S. Reclamation Service have been subsidized at least to the extent of 40% (the ratio is apparently more than 70%, according to Mr. Lippincott). In addition, considerable capital and operating expenses have been cancelled. That is a sad picture which, after twenty-seven years of effort, can only be described as direct financial failure. Some of the causes can be remedied, but others will continue for many years, and the conclusion seems inevitable that under present conditions Government effort in irrigation cannot be made profitable in a literal sense. Similar effort in the immediate future will involve heavy financial subsidies. The case for the immediate extension of irrigation on a considerable scale will have to rest on the indirect advantages, particularly those that are expected to accrue after a longer period even than a generation or two.

Throughout the world irrigation on a large scale has only been immediately successful under two essential conditions: (1) When the physical aspects are such that the works would be exceedingly cheap; and (2) when there is a large and frugal population, crying out for agricultural expansion because the opportunities for their absorption into other more profitable forms of occupation are restricted.

India fulfills both these conditions. Spain and Italy fulfill the second, and, in regard to some of the earlier works, the first condition; but in later works both countries have found it necessary to resort to subsidizing. The density of population in America is scarcely one-third that in Spain and one-sixth that of Italy; the relative density of the agricultural population is still less; the standard of living is far higher; and towns and industries continue to offer a more attractive occupation. The problem of attracting settlers to irrigated lands is only a part (the more expensive part) of the general problem of encouraging agricultural settlement and development; and as long as the Government attempts to force the natural tendencies of a population it is likely that subsidizing in some form will be necessary. Judging by results throughout the world and by the standard of direct financial success, it seems as if the United States has been premature in anticipating a genuine land hunger. South Africa has been still more premature in attempting to recover the entire expenditure, with interest, under much less favorable physical and economical conditions and an even weaker intensity of land hunger.

Without entering into the question as to whether it is desirable that irrigation should be pushed ahead by Government, even if it involves an immediate direct financial loss, it should at least be pointed out that in such countries as Spain and Italy, where irrigation is an old problem, the greatest prosperity is generally to be found grouped around the old irrigation works. The road to this prosperity has seldom been an easy one. Safety and security have often been reached only by a process resembling that by which the soldiers of Cortez crossed the watery gaps in the Mexican causeway over the dead bodies of the advance guard.

Assuming that a Government has decided to continue irrigation developments with subsidies there are a few aspects of policy that might be added to those presented by the Committee. These fall under three heads: (1) a steady program of reconnaissance and construction; (2) subsidizing by additional expense for development and settlement in preference to, or prior to, reduction of expenditure on works; and (3) taking the task of rate collection completely out of the range of politics by placing it in the hands of private institutions or banks.

Steady Program.—It is unfortunate that the popular demands for new irrigation developments generally come like sudden devastating floods and they often synchronize with periods of agricultural depression or political elections. They prevent an adequate preparation; they upset all economical arrangements as to staff and construction plant; they cause land to be brought under irrigation before the advent of a steady flow of settlers; and they are apt to disturb market conditions. It is most important that a more or less steady program should be arranged over a long period. Furthermore, there should be no relaxation in the forces of reconnaissance, so that the necessary investigations may go on steadily well in advance of the program of construction. These considerations are more important for irrigation developments than for any other form of public works because essential factors, such as hydraulic and meteorological measurements, can only be properly assessed after long periods of observation.

Subsidizing Developments.—The remarks made by the writer in 1913 as to the slow rate of development and settlement seem to be equally true to-day (1929). If an irrigation project could be built to deal with a slowly expanding demand with the same facility as in the case of railways, roads, telephones, etc., the slow rate of development would not be such a serious factor. Unfortunately, in most irrigation developments, the major part of the ultimate expenditure on works must be incurred at the beginning. A simple study in arithmetic of compound interest and accumulating overhead charges will demonstrate quickly the absolute need of hastening the full development of the land affected by the new irrigation works. Experience has shown that under the existing arrangement (which relies on the private capital of comparatively poor people) development requires at least three times as long as it would with all the necessary capital available at the beginning. Even waiving the interest on capital expenditure for works has not proved sufficient to draw from private sources the capital needed for rapid development on such lands.

If a business that can hardly meet current interest and overhead costs after 20 years can be made to reach the same stage after 5 years, there is a considerable sum (corresponding to lost interest and overhead expenditure during 15 years) that could be utilized better as a free gift in the beginning to accelerate the development. The case becomes even stronger if the slow development of the land under conditions that now exist is not on the soundest possible basis and if the Government, with experienced and trained workers and capital resources readily available, can achieve better development as, for example, in leveling the land, which will result in a greater return from the water made available by the capital expended in the works. It seems preferable, therefore, to give the first subsidy in the form of free capital for accelerating development of the land, but to leave interest charges on at least part of the capital cost of the works so that its accumulation and excessive overhead costs may be a continual spur toward accelerated development. There will be complications in dealing with those who have already developed land from their own resources and in seeing that an early settler does not take all the advantage at the expense of a later purchaser; but there are complications of this kind in every form of Government bounty.

Collection of Rates Through Banks.—There is nothing so damaging to the financial credit of irrigated areas as the deferment and accumulation of rates that form a first claim on the land. A common failing of certain politicians is to endeavor to gain immediate favor with their electors by exercising the utmost pressure to obtain these deferments. By placing the collections in the hands of banks this kind of political interference will be considerably reduced. Stern action on these lines will also be the best cure for inflated values of land and for the undesirable survival of unfit settlers as well as speculators. These are all very prevalent evils under existing conditions. Moreover, bank management will tend to dampen the activities of those who press for the immediate construction of new projects imperfectly investigated while relying on similar pressure at a later date to evade payment.

Joseph Jacobs,[57] M. Am. Soc. C. E. (by letter).—It was during the writer's term as Chairman of the Irrigation Division that the Committee on "A National Reclamation Policy" was created, and his letter to the Committee, at the time of its appointment, included the following paragraph:

> "In deciding to create this new committee the Executive Committee felt that reclamation matters, particularly in respect of State and Federal relationship thereto, were just now in a state of transition and uncertainty, and that a correct solution of what the governmental attitude should be, required the careful consideration of able men entirely familiar with the subject of reclamation. This is a matter of great economic importance not only to the West and South, where reclamation activities will be greatest, but to the nation as a whole, and it is believed that no other group of men are better equipped, nor indeed anywhere nearly so well equipped, to contribute constructive thought and analyses of these matters, for the advice and guidance of Congress and of State legislative bodies, than is a group of carefully selected engineers, such as is represented by this committee."

That the Executive Committee's apprehension at that time was well founded is evidenced in a growing restlessness concerning reclamation matters and a steadily increasing spread of propaganda directed toward discrediting and discouraging further Federal reclamation activities. So pronounced has this trend been that, on August 26 and 27, 1929, a Western States Governors' Conference was held in Salt Lake City, Utah, to review the situation. However much or little one may agree with the findings of the Committee report, it is gratifying to note that in the solution of important economic public questions which have distinct engineering correlations, engineers specially qualified for the task are willing to contribute their time and their trained judgments to the analyses of such questions, solely for public benefit. This, of course, is as it should be.

The writer finds himself in general, although not in complete, agreement with the findings of the Committee. While these findings have been set forth in clear, unmistakable terms, each of them is so important as to have merited some accompanying discussion and supporting data, and the value of the report would have been further enhanced by a prefatory review of the Federal Government's past irrigation activities, its problems, and its accomplishments. Mr. Lippincott, in his subsequent explanatory statement concerning the report, sensing this curtailment, supplies many of these missing data, and it is to be hoped that the Committee can incorporate some such statement as an integral part of the final report.

---

[57] Cons. Engr. (Jacobs & Ober), Seattle, Wash.

The writer cannot accept, without qualification, the implications of Principles (9) and (10). It seems to him that the Committee is unduly exercised over the bugaboo of over-production. With a present annual importation of $800 000 000 of foodstuffs, and with a population growth of 1 500 000 per year, over-production in the United States, if it be a fact, cannot long continue and, in any event, it constitutes no sufficient warrant for long delaying the commencement of those major projects which cannot possibly be brought into full yield short of 20 or 30 years. Moreover, agricultural production is not wholly a National problem. It is, to a considerable extent, a regional problem also, and the desirability of an irrigation project the products of which may have, largely, a local distribution, should not be judged too rigidly by National considerations of possible over-production. Local economics must be accorded its due weight in reaching correct final judgments.

Principle (10) declares that the State should share in the responsibility for the selection and approval of projects, and the writer would add that the State should also share in the financial responsibility for the project. State co-operation should be made a condition precedent to the extension of Federal aid. The State's interest in reclamation is greater and more immediate than that of the Federal Government, and second only to that of the settlers on the project. All these agencies should co-operate. If a State, through lack of courage and enterprise, or because it has not sufficient interest or faith in its own development, is unwilling to assume part of the responsibility for that development, it is not entitled to Federal aid. It has no moral right to ask the Federal Government to assume, on behalf of the State, a responsibility which it (the State) is not willing to share. If, in some instances, present State Constitutions inhibit a practical financial co-operation as here contemplated, this can be remedied by Constitutional amendment. A well-planned co-operation should result in more carefully selected projects, in a saner and more rapid settlement, which is all-important, and, finally, in safer project investments for all concerned. If States deem such Federal aid and co-operation of value, they will soon correct any Constitutional defects that stand in the way of their securing it. The formulation of the details of a practical co-operation, of course, will require careful consideration.

Many, no doubt, will take exception to the Committee's condemnation of the waiving of interest on construction charges (Principle (11)). Some substantial reasons can be advanced for the interest waiver, particularly as part of the original Reclamation Act with its otherwise too drastic payment terms; but the writer believes that the weight of argument, and sound economics, favors the stand taken by the Committee. The only financial concessions that irrigation needs from the Federal Government are a low rate of interest, long terms for payments (not less than 30, nor more than 50, years), and the absorption by the Federal Government of the interest charges accruing against unoccupied lands during the period of settlement. The last-named concession may be regarded, not as a subsidy, but as a payment by the Federal Government for its share of the benefits resulting from irrigation development, and the Federal benefits from such development are by no means inconsequential, whether considered from the social, industrial, or military viewpoint.

With these concessions, any project worthy of development should be able to meet its obligations as they mature. Not only is it the writer's opinion that the projects can pay, and that it is just that they should pay, a low rate of interest on terms as specified above, but that, unless the Western States are willing to accept such a consummation, Congress will soon refuse approval of any further extension of Federal reclamation beyond present commitments. If such acceptance by the Western States was not justifiable on any higher grounds than expediency, it would have ample, practical justification on that basis alone, because the return to the Government of its investment with a fair rate of interest is probably the only basis upon which land reclamation can be depended as a continuing Federal policy.

The authorities responsible for a National reclamation policy, as Mr. Lippincott points out, should have in mind the creation of equal opportunities and the stimulation of individual effort; but they should also have in mind that reclamation must be founded on so sound a financial basis that the return of construction costs, plus a reasonable interest charge, is assured. Otherwise, it is not enduring and, sooner or later, must inevitably forfeit Congressional support. It should also be sufficiently broad and adaptive to contemplate the needs of the entire country. People of the West are too prone to interpret reclamation as meaning only the watering of arid or semi-arid lands. From a National viewpoint, however, and this is an all-important viewpoint, the irrigable lands constitute only a small part of the reclaimable lands of the United States. About 25 000 000 acres have already been reclaimed by irrigation, and it is estimated that a like area may yet be served economically by water. The swamp-land area, however, approaches 100 000 000 acres, and the logged-off land area is practically the same. The remaining irrigable area, therefore, represents scarcely 15% of the remaining reclaimable lands.

They of the East and the South, where the major part of these unreclaimed lands lie, are just now (1929), more than ever, voicing the sentiment that the Government's future reclamation policy shall not concern itself solely with arid lands, but shall embrace these other lands as well. It is pointed out that, whereas the Government has expended millions (about $185 000 000) in aid of irrigation, practically nothing has been done toward reclaiming the swamp and cut-over areas; that the cost of reclaiming these areas would be no more, and generally would be less, than present costs of irrigation; and that they lie nearer the centers of population and, therefore, could be more readily colonized. These arguments appear plausible and may be sound. They merit investigation and consideration in formulating a National reclamation policy.

There has been some suggestion in recent years that the Federal Government confine its activities to the construction of storage works and, on the theory that these are required for flood control, that the Government assume all construction and operating costs, and make the waters thus stored available, without charge, for irrigation and other uses. (This should not be confused with President Hoover's tentative plan, as presented by the Secretary of the Interior at the Conference of Governors at Salt Lake City, on August 26, 1929.) So far as irrigation is concerned, the suggestion is subject to the following objections:

(*a*) In some cases it would be a fictitious basis for Federal aid, for there are districts where there is serious need for irrigation storage and little, if any, need for flood-control storage.

(*b*) In many instances storage may represent but a small part of the development cost. The Columbia Basin Project, for instance, and such projects where magnitude or other conditions impose a necessity for Federal aid, would never be built if the National Government could construct only the storage works.

(*c*) Unless the stored water is charged for (and this charge, if made, should be actual cost) its allocation can become a matter of political favoritism and political spoil. Allocation to various agencies on the basis of priorities, needs, and benefits, and with a fair charge for water on long-time contracts, is more rational and business-like than supplying free storage water. The cost of that portion of the storage works that is properly attributable to needed flood control should be borne by Federal and local governmental agencies; that portion which is necessary for the improvement of navigation should be borne solely by the Federal Government; and those portions which are for the benefit of domestic, irrigation, and power uses should be paid for ratably by those agencies. A fine business integrity is thus preserved, and unnecessary paternalism avoided.

(*d*) To charge nothing for storage water makes the Federal Government figuratively and actually a "wet nurse" to irrigation projects, and that is good neither for the Government nor for the project. A worthy project can, and is willing to, pay for its necessary water supply.

F. N. Cronholm,[58] M. Am. Soc. C. E. (by letter).—No matter how business-like and favored a new reclamation policy might prove as to return of capital invested in irrigation works, the fact remains that settlers who chance to acquire good land which, because of its physical conditions, can be quickly and economically farmed, will prosper; and the settlers not so fortunate will fail. As long as the study of the problem is narrowed or confined so that it does not go beyond bringing water to each farm unit, there will be little, if any, improvement. Probably no one knows the relative cost per acre of making lands ready to seed as against that of bringing water to the farm. It is reasonable to assume that the cost is more per acre to bring a farm to the same plane of perfection as the irrigation works, and if this is approximately the case, the writer feels that the settlers' viewpoint on the problem has not been given as full consideration as it should have. Certainly, the magnitude of this phase of reclamation would seem to warrant greater analysis.

The writer believes that more should have been done as regards Government reclamation projects than merely bringing water to the highest part of a farm unit. If it is reasonable to have the works built by the Government as efficient, as lasting, and as dependable as they are, then it seems reasonable that the lands served be as well provided. Truly, they are inseparable; so if there is justification for excellent dams, diversion structures, canals, and laterals ready to serve, then there is like justification for the canaled area leveled and, in every particular, ready to farm.

What the U. S. Bureau of Reclamation needs, if it does not already possess it, is a policy enough ahead of the times so that when a project is completed everything about it will be attractive. An irrigation project is, after all, although indirectly, a kind of plant intended to produce food, and once it is commenced it is not complete until the land is made ready to farm. Directly from the viewpoint of the State, as well as that of the Federal Government, the main purpose of a project undertaking should be the building of a community of happy, wholesome people.

---

[58] Los Angeles, Calif.

There are vast areas of desert already peopled to the benefit of both State and Government, and such work by all means should continue. There are also vast areas elsewhere, long since settled, that are dying or that are already dead. It should be the business of the Bureau of Reclamation to plant an example, as it were, here and there throughout such localities and revive crop output by working the land to capacity, making the community idea of settlement so attractive that people will abandon other places that are hopelessly isolated and impossible to live in happily or to farm profitably. The cost of such community building should be pro-rated among those benefited, namely, the settler, the State, and the Federal Government. Make the sum to be refunded by the settler such that he can pay both the principal and interest and always feel decently cheerful about his contract as compared with what he might have done elsewhere.

What about the vacant areas of existing irrigation projects or extensions? Before they can be disposed of must these lands not be made attractive? Truly, there is more involved than the reasonableness of the annual maintenance and the repayment of construction charges. The prospective farmer will estimate, as best he can, to what extent he will be obliged to apply what monies he has toward getting the land ready to produce. Whatever the outcome, there is no sound logic in permitting "Tom, Dick and Harry" to hazard monies invested by the Government by allowing them to level land, even at their own expense. The preparation of the surface soils for the application of water as done by settlers has, in many cases, been against best crop output, against economic application and use of water, and a permanent detriment compared to what would have been the case had leveling been rightly done in the first place.

For instance, consider the case of a farm unit on which the surface soils happen to be more productive than those of greater depth. These rich soils are likely to be wasted by finding lodgment in hollows, whereas they should be saved to spread over the surface as the farm unit is leveled. However, the settler is much like any other person. He is apt to overlook many things; in fact, he may be obliged to do so because of the necessity of producing something before he "goes broke". Time is a factor. He will eat up what he has before harvest time if he does not hasten planting. Therefore, saving top-soil becomes secondary as does the methodical preparation of the surface. The settler plants first probably an already level patch, irregular in contour. Water is brought to this naturally level piece of land by short-cutting across the farm unit—meandering around this hollow and that bump. He has now permanently disfigured, as well as destroyed, his unit for future maximum economic crop output.

Water is brought to land for a purpose, and it is the purpose in which the entire nation, as a people, is primarily interested. Therefore, while water is indispensable in arid regions, it is but a means to an end, and this objective should be the same in Alabama as it is in Idaho. In the beginning the purpose of the green spots to be created by the U. S. Bureau of Reclamation in the deserts of the Western States was, it is stated, for food production in given areas, and at the time this was assumed as ample reason for expending monies for irrigation works; possibly so. At any rate, it brought about a beginning by creating a department in the Federal Government that should continue in the good it is doing and can do. The objective of food production of long ago must be enlarged upon, however, or changed, in order that it may be more Nationally adapted. Make this purpose as applicable to Alabama as to Idaho. Let the objective be community building, reviving of soils, and promoting the interests of the settler. Do work in every State for the purpose of bringing up the average. Make irrigation works, stump pulling, land leveling, draining of swamps, or adding fertility to soils secondary in so far as they are but a means of bringing about the objective. Preparing lands to the point where they are ready to farm and offering them to people at a price and on terms that are attractive and well within their ability to pay is an undertaking that—coupled with assistance and guidance needed in the matter of suited crops, market outlets, etc.—would provide a greater certainty of return of investment to the Government than when the Government undertakes to do but one of the major necessities, such as bringing water to land. The program should include any assistance or guidance needed in the matter of choosing suitable crops, market outlets, etc.

Converting rough, raw land into producing farms has been done by settlers poorly fitted, both as to knowledge and money. Poor and humble as many of them were, each one must have believed that his progress would be forward. As a matter of fact, many people lost their entire savings and quite permanently wrecked themselves. Every one having a part in this first work was forced to a lower standard of living for quite a time and, as a result, no one of them is inclined to bestow gratitude. If it costs from $60 to $120 per acre to bring water to a farm, it costs on an average about as much again to prepare land ready to farm, that is, with the understanding that the work is done as lavishly and completely in one case as in the other. Therefore, it seems that the settler, in preparing his land to farm, even though inadequately, has certainly done his part, whether it was on a Federal or a private enterprise.

To expect private enterprises to solve the problems of Federal Government undertakings is of no use. Policies and methods of merit, differing from those applied by the Government, have always been available and would, undoubtedly, have been adopted forthwith in the beginning had such policies and methods been most adaptable. The writer has observed that the Government has always been progressive. The men responsible for making decisions since the infancy of the Reclamation Service are to be commended, and, considering the magnitude of the work accomplished, have an enviable reputation in that they made so few mistakes. The Government's most questionable projects are poorly located from the point of view of marketing and climate. The idea that the more private a project is, the more successful it will be, or the more public it is, the more hazardous it will be, should not be taken too seriously. Take Imperial Valley as an example. This project is what might be termed "half and half" since it is not entirely Government and not entirely private. It has ample area so as to be representative; it has many sunshine hours, exceptional growing climate, and fair marketing conditions. At that, it has seen many "ups and downs" and most certainly, it is not yet "out of the woods". Comparisons are to no end and, as previously stated, of no value.

What irrigation projects lacked at the time the Government began its work was scope. Irrigation practice, outside of possible district projects, was strictly commercial, first, last, and all the time. Works were built with total disregard to future upkeep, and it was the settler who shouldered the burden of paying the cost as well as a profit to the promoters. Salesmanship was a main feature because good and lasting works could be slighted or needed features could be eliminated altogether, depending on the amount of sales talent. As a consequence, people settled the areas more out of the hope and glowing promises engendered by high-powered sales talk than because of any material thing of worth. Re-sales resulted, and the next settler was able to benefit somewhat by the labors of the man he succeeded. While this may be irrelevant and immaterial and of no moment, the writer believes that if private and district projects had gone another step toward making lands ready to farm and had been leveling land when the Government entered the field, the Government would have adopted such a policy also and, to-day, all projects would be better off financially and their settlers more prosperous.

Perhaps the most serious lack was the foregoing, namely, the absence of land leveling as a practice in project construction. Besides there was the mistake of using too generously, in estimating acre costs, the lands within the reach of the projects' distribution systems. Many of these lands, on detailed examination, would have been excluded. The suitability of the rougher lands, from the viewpoint of the prospective settler and his ability to pay, was not studied as completely as it should have been. Fitness of lands to farm should have been given the same study as the building of irrigation works.

It is all right in 1929 to say that twenty-five years ago project settlers should have been only those with $5 000, or more. That was considerable money in 1904, and people who moved about in covered wagons were, for the most part, hard-working folk glad of employment in the building of project works. Often it was these same people who, later, out of hard-earned savings, became project settlers. Such people are gradually disappearing like the buffalo and the antelope.

The only solution to land settlement projects, whether or not the question of water is involved, or whether the work is done by the Federal Government, or by private or district organizations, is to do more toward making lands ready to farm. Even to-day people with $5 000 or $10 000 are likely to wear too good clothes for farming. What, in fact, is there against helping, or rather, creating opportunity for, the man with $2 000, or less? There are many such worthy people, and, generally speaking, if the Government is interested in social uplift and community building through giving impetus to crop output and organizing marketing, then it is these people who have the greatest need.

If there is ever a time when a man needs all his allotted land in shape to farm, it is in the beginning. For instance, if in a given locality it will require, ultimately, say, 80 acres to support a family in an American way of living, then it is imperative that the entire unit be ready to seed as soon after settlement as growing seasons will permit. The size of a farm unit should, in any case, be sufficient to support the average family properly, and allow for growth and education. Decent, respectable living with forward progress contingent on honest effort can rightfully be expected by any man willing to work to improve his condition, and under no circumstances should the Government lend itself to an undertaking that would hazard such progress. Most certainly it did this during the early project settlement and, in fact, does so now every time it permits a settler on raw land not ready to seed. Such risking of a man's chance is diametrically opposed to what the Government's purpose should be.

In spite of the foregoing, the U. S. Bureau of Reclamation ranks high in accomplishment. There is no organization in the nation that is cleaner, or that can show more good per dollar invested. The Service must become more human and more National in scope. It must do reclamation work that will draw people from various States to a given locality for a specific purpose and thus create new life and bring hope nearer realization. At present, the problem is, what to do with unsettled parts of existing projects so as to hasten their settlement. The writer suggests the following as a beginning:

1.—Choose a project having unsettled land that is so situated geographically that additional crop output could be profitably marketed.

2.—Furnish maps showing the topography of each farm unit in the rough and others showing how the same land will look when it is leveled, ditched, and diked, and the necessary control structures are in. Show these maps to the prospective settler in order that he may see what is before him as well as choose what particular unit of those available he likes best (allowing progressive settlement only).

3.—Level, not part of the farm, but all of it; and do this following the most approved method according to soils, in a manner allowing for a reasonable range of crops. Lay out the work so that the farmer may manage without hired help if possible. Keep in mind economy of water for the good of crops.

4.—Besides leveling the farm, turn the water on to it; if, in wetting, any areas settle, re-level that area. Make sure the entire unit is ready to plant. Furthermore, provide outlet for surplus irrigation water and, in the farm layout, give some thought to possible future ground-water relief should that become necessary.

5.—Do not prepare lands to seed until the area has been applied for by a qualified settler who shall be required to put all the land in crop beginning with the first growing season and keep it in crop for two full seasons before he is allowed to file on the farm unit.

6.—Have each prospective settler acquire the necessary credentials to become a conditional settler from a local board entrusted by the people of the project with determining an applicant's fitness, the board not being limited by a prescribed minimum money requirement.

7.—Collect in advance project maintenance charges. Add the cost of leveling and otherwise preparing the farm unit to project costs against the land, and of the total take a fair amount to be amortized in a reasonable time by the settler. Have him make the first payment and interest on deferred payments prior to the third season and yearly thereafter without fail. Make the amount to be ultimately repaid the Government of such size that the farmer will not fail because of it, but, at the same time, make it an obligation generally conceded as ample.

8.—Do not charge for water used during the first two seasons. If the prospective settler pays all other expenses involved in cropping the entire unit twice, and makes ready to begin paying off the project debt by the third year, he will be doing well. After all, the would-be farmer will compare every location available to him, weighing advantages against disadvantages, before finally deciding on a particular project and what it has to offer. Once a man has become involved in a proposition, it is well that he continues to be satisfied.

9.—Build no project without the prior approval of the State, or States, within which it is located, and then only on condition that the State, or States, be interested enough to guarantee the Federal Government the full amount of estimated costs in case of the settler's failure to meet the terms of his contract.

The foregoing list resolves itself into a plea for "ready-to-farm lands" and would involve a great deal in the matter of organization, although, presumably, the educational work needed for its authorization and the procuring of needed monies are the more difficult of accomplishment. It is acknowledged that something must be done to settle lands on Government projects now vacant. Personally, the writer cannot help feeling that preparing farm units ready for seeding would bring sufficient settlers of the right kind to these areas.

In further clarification of Suggestion No. 7, it seems possible and equitable to the farmer as well as to the Government to fix a rate of interest on deferred payments that will, in the end, make up the amount written off in the beginning so that, as a matter of fact, the Government will be repaid in full. In this way the Government would not be a money lender charging rental, as it were, for its money, but the farmer would be paying interest nevertheless. For instance, say the project costs $150. The Government would only charge the farmer $75, but it would charge him interest at a rate which, in a certain term of years, would ultimately repay the total project cost to the Government.

If something along the lines suggested could be authorized, the writer feels sure that it would succeed. Making lands ready for seeding should be done for the Southern States as an experiment, and with the two demonstrations, one in the West and one of fair acreage in the South, the writer believes that an adaptation of the U. S. Bureau of Reclamation to greater National good could be attained, and, in turn, the objectionable political phase lessened. The good the Service would then be doing would be more broadly understood and welcomed.

R. I. MEEKER,[59] M. AM. SOC. C. E. (by letter).—The National Reclamation Act of 1902 was framed with the purpose of using monies from the sale of the public domain in the arid and semi-arid West for the reclamation of other arid lands. Under the Oil Leasing Act of 1920, 52½% of the income from oil and minerals from public lands was added to the National Reclamation Fund, and, under the Federal Water Power Act of 1920, 50% of income from that source was also added to the Fund. Federal reclamation to date, therefore, can be stamped as Western aid to Western development since Western resources have been used to finance irrigation development.

Income from public lands in the arid States is dwindling. Future irrigation projects as outlined by surveys will be large and costly and with a much greater unit cost per acre than during the twenty years since 1910. Many proponents of reclamation are advocating greater Federal participation. The supplemental report of the Committee mentions the need of National aid to reclamation by direct appropriations from the Federal coffers for public works for irrigation—this in addition to the present Reclamation Fund. The precedents of Federal aid for navigable rivers, highways, aviation, flood control, and inland waterways are cited as the arguments for such National aid for future water utilization.

If such procedure is imminent, and it so appears, then a new phase of financing arid reclamation is in the offing, and Federal reclamation will be more than a term; it will be a fact, and the relations of Federal and State activities in water matters will assume a matter of extreme importance to the Western States in view of past experience with Federal reclamation.

The desire for "Federal aid at any price" (which has been so rampant in recent years), and at the price of local government, is not believed to be advantageous to the West.

Federal and State relations and co-operation in water matters are specifically referred to in Items (3) and (7) of the Committee's report, and one page of the supporting statement is devoted to discussion thereof. The writer desires to direct attention to an unenviable feature of past reclamation and water utilization with the hope that future Western reclamation may be devoid of a repetition of such offenses, especially if National funds are to be the backbone of such development.

The ownership and control of Western rivers lie in the Western States as announced in constitutional provisions and enabling acts of the various States, and as sustained in the Kansas *vs.* Colorado, U. S. Supreme Court decision. Paragraph 8 of the Federal Reclamation Act specifically states that Federal reclamation shall conform to and be subservient to State water laws. The history of National reclamation does not chronicle complete acquiescence to State laws in such matters.

The National Reclamation Act was scarcely passed before the U. S. Reclamation Service intervened in the U. S. Supreme Court water suit between Kansas and Colorado concerning the Arkansas River, claiming National title to and right of control of the waters in non-navigable streams of the Western

---

[59] Cons. Engr., Denver, Colo.

States. Furthermore, the Reclamation Service asked the Court to announce a doctrine of priority in water matters regardless of State lines and irrespective of whether the riparian or appropriative doctrine obtained in adjacent States. The Supreme Court held adversely to the Reclamation Service and confirmed the rights of the States as to complete jurisdiction over their rivers.

The entry of the Federal Government on river systems of the West immediately chronicled the commencement of Federal interference in State water matters and opposition to proposed private power and irrigation projects even where the rights of such projects in many instances were anterior to the rights of Government projects. Illegal and injurious embargoes by the Federal Government were imposed on some of the river systems for the so-called protection of Government projects. Canal and reservoir rights of way were denied by the Department of the Interior at the instigation of the Reclamation Service on the grounds that the construction of such projects would jeopardize the water supplies of the Federal projects, despite the protection under State laws of their rights by the doctrine of priority. Not satisfied with priority dates of Government projects, the Reclamation Service blocked and "killed" power and irrigation projects, especially reservoir features thereof, within river systems where Government projects were located. Trans-mountain diversions through the Continental Divide were opposed and defeated by denials of rights of way by the Department of the Interior on the flimsy ground of injury to the Yuma Project on the Lower Colorado River, etc. Hence, while the Reclamation Service and affiliated Federal bureaus had no direct control over Western rivers, they maintained indirect control of parts of river basins and water supplies through control of Government land, giving preferential status to Federal projects.

International disputes were unfairly invoked to the detriment of the development in river basins in the United States. A list of private projects blocked and "killed," with the means resorted to in "killing" them, would not grace the present Federal accomplishments.

Even at this late date Federal reclamation officials are claiming ownership of return flow and drainage water from Federal projects with sales of the water to other projects lower down on river systems. The U. S. District Courts have become a refuge for Federal projects in return-flow disputes, and a line of U. S. District Court decisions is being forged favorable to Government ownership of return-flow waters.

Federal bureaucracy has left an unnecessary trail of blighted hopes, blocked and "dead" power and irrigation projects, now submerged in the rainbow of accomplishment. Section 6 of the National Reclamation Act specifies that ultimately the management and operation of Federal irrigation works shall pass to the water users served thereunder. This local management is gradually coming to pass, but all too slowly. Early local management and operation of Federal projects will go a long way toward the elimination of Federal interference in water administration. The assumed preferential status of Federal projects in water matters will then collide with State laws, and it is hoped that such assumed preferential status will be effaced.

Item (15) of the Committee report, *in re* turning of reclamation projects over to local management at an early date, is highly desirable in order to get away from a part of Federal interference and opposition to State water laws and river administration.

The writer realizes that the encroachments of Federal reclamation on State water control and the special privileges assumed by Federal projects have been made possible partly as a result of local encouragement by politicians and water users under some of the projects, and even State legislators who were willing to go the limit in getting Federal monies expended on their river systems. This situation still persists as to proposed Federal projects. The expenditure of large sums of money from the National Treasury on reclamation and water utilization in the arid West should not be looked upon with favor unless State and Federal relations are clearly defined, and abuses of the past are avoided.

The August, 1929, proposal of President Hoover authorizing the transfer of the remaining public lands to the Western States, and voicing a hope that a line of action might be followed that would "check the growth of Federal interference in affairs of essentially local interest, and thereby increase the opportunity of the States to govern themselves," is a consummation devoutly to be wished in water matters of the arid West.

The writer heartily concurs in Item (8) of the Committee report to the effect that future stream regulation shall give preference to supplemental water supplies for existing Federal or private projects before new projects are considered.

E. B. DEBLER,[60] M. AM. SOC. C. E. (by letter).—The Committee proposals, briefly stated, comprise:

(*a*) Stream development in accordance with general plans formulated by the Federal and State Governments with basic data obtained by them.

(*b*) Change in Federal aid from that of waiving of interest to the assumption of part of the cost with low rate of interest to be charged on the remainder.

(*c*) Finding of feasibility and recommendation by experts on all phases to be a prerequisite to the approval of new projects for construction, with the States to share the responsibility for the selection of projects; the elimination of State lines and local interest in such selection.

(*d*) Construction preference to be extended to projects providing supplemental water supplies, and the reclamation of new lands to be delayed.

(*e*) Construction repayments to commence promptly upon the completion of each unit of the project, to be initially light, and to extend over a long term, with forfeiture of land upon delinquency.

(*f*) Aid in settlement to be provided by States and local interests.

The lack of an official discussion by the Committee, as a whole, of the annunciated principles leaves a wide latitude of interpretation for the reader. The writer has necessarily relied largely on Mr. Lippincott's discussion for guidance in this situation.

---

[60] Hydr. Engr., U. S. Bureau of Reclamation, Denver, Colo.

The desirability of planned stream development needs no defense; but acceptance thereof by States will not always be readily obtainable, as witness the wrangling over the Boulder Canyon (Colorado River) and the St. Lawrence River developments.

Change in Federal Aid.—The proposed change in form of subsidy from a waiver of interest on construction cost to one of outright donation of a part of the necessary works, provided the remainder of the works are constructed by other agencies, has one very decided advantage over present conditions. It obviates the always unpleasant task of collections which, in the nature of things, is always difficult when anything is bought under the instalment plan, whether it be a radio or a water right. Mr. Lippincott at one place advocates that Federal aid take the form of reservoir construction on navigable and interstate streams, with similar aid on non-navigable streams to be provided by the States. Such a course would discriminate heavily against the inter-mountain States having no navigable waters. On the other hand, States fortunate enough to be possessed of navigable waters would receive a double benefit of subsidized water supply and subsidized transportation. If such a plan were enacted, navigation on mountain streams might suddenly be found to have surprising possibilities. Mr. Lippincott appears to recognize the exceptions that would be necessary in that he considers it only fair that a Federal appropriation be made for the construction of the All-American Canal. While storage construction would go far toward solving Sacramento Valley problems, it would be only an insignificant aid to the Columbia Basin Project. In justice to the Committee as a whole it must be noted that the report is not specific on the development features to be donated by the Federal Government or the basis on which appropriations for such work are to be secured. No doubt, a plan can be evolved which will not favor particular sections. Success will be greatly enhanced by general applicability.

Viewed in the light of the situation which would arise in the event that the public lands and minerals are ceded to the States, with elimination thereby of the original source of the Reclamation Fund, the Committee proposal merits careful consideration, especially by the Rocky Mountain States. Agriculture in the Lower Mississippi Valley is receiving a heavy subsidy in flood protection and cheapened water transportation. The Panama Canal gave Pacific Coast agriculture a decided advantage over the mountain region in reduced freight rates, while greater advantage is being sought under the guise of local aid to navigation.

Finding of Feasibility.—Careful investigation of new projects, as recommended by the Committee, will not mark a new departure for the Bureau of Reclamation. In earlier years, more stress was laid on engineering than land classification and economics, and rightly so, as works of a new magnitude and difficulty of construction were necessary. Agriculture was not then, nor for many years afterward, in a depressed state. With few exceptions the lands served with water could, at moderate cost, be placed on a profitable production basis.

The most unsatisfactory feature then was the inability to compel participation in costs by privately owned lands, the holders of which needed to pay only when using water. Irrigation districts with their powers of general assessment were introduced to meet this situation.

The agricultural situation of recent years and the less desirable quality of much of the land remaining to be irrigated have necessitated greater attention to matters of soil, land classification, and economics, so that these factors are now given consideration on a par with engineering. Considering the magnitude of the works undertaken and the human difficulties encountered, the results on Government projects compare very favorably with those obtained under private activity.

If projects had been accepted purely on their relative merits, the results to date would no doubt have been more favorable and the prospect of successful future development much brighter. The Bureau of Reclamation has not endorsed some projects undertaken in the past, and Congress has seen fit to modify the adopted 10-year program. Agitation for construction of projects without regard to their prospect of success has never been more pressing.

The proposal to have States assume responsibility in project selection has been tried by the Bureau through representation of the States on boards that reported on projects, often with disappointing results. Lack of training and, in some cases, exertion of political pressure and business influences precluded intelligent and unbiased consideration.

Construction Preference.—The suggestion by the Committee of favoring projects that will provide for supplemental water supplies for existing developments is, in fact, already the policy of the Bureau, and with this policy Congress appears to be in harmony. Projects of this kind, when planned with due regard to proper stream utilization, will often result in the development of new lands.

The Gooding or Minidoka Gravity Extension Project is of this type. It was started as a Carey Act project more than twenty years ago, when a reservoir was constructed on Big Wood River and canal systems were built to irrigate the greater part of an area of 200 000 acres. Lack of water led to gradual shrinkage of the district area to 76 000 acres, of which about one-half is cultivated in scattered tracts, with a water supply inadequate for profitable farming. The only practical solution is to bring a supplemental supply from Snake River by means of a long, expensive canal to deliver stored water from American Falls Reservoir and, incidentally, provide the means of irrigating an area of new land as large as the old. While it is generally desirable to delay settlement of new lands (in this case largely public in character), the result is to load present settlers with the operation and maintenance cost of works intended for a much larger area, construction to less than full capacity being impracticable.

Suspension of Construction.—The agricultural depression of recent years has so seriously undermined the financial stability of innumerable privately constructed developments, especially those with inadequate water supply or expensive operation, that wholesale abandonment can only be averted by reconstruction which they can not finance themselves and in the course of which some new lands will almost invariably be supplied with water. A general policy of discouraging, temporarily, construction that tends to increase crop production, is advocated by some as desirable. If adopted, the policy should not be limited to the arid states, but should be extended to all activities that tend to increase production, such as flood protection for overflowed lands in the Mississippi and Sacramento Valleys, drainage of swamp lands in all non-arid States, development of cut-over lands, and the rehabilitation of exhausted or eroded lands of the Eastern States. Such a policy requires careful consideration of the speed with which serious shortages in food supply can develop, while the preparation of new areas for production consumes much more time, especially in the case of irrigated lands; neither should a purely altruistic attitude of "the greatest good to the greatest number" be adopted in preparing the policy; the relative needs of various localities for development to maintain and improve their existence should be given careful consideration.

The continuation of development with the Reclamation Fund can hardly be considered a tax on other communities since two-thirds of the funds now used are derived from repayments by constructed projects, and this proportion is on the increase.

Construction Payments.—The Reclamation Bureau policy on the whole has never been other than that proposed by the Committee, although individuals within the Bureau, and possibly department heads, have wavered at times while certain members of Congress have publicly fostered resistance to such a policy and by every means at their command have aided and abetted evasion and reduction in payment of monies due the Government.

The crisis of 1921 and the subsequent years of agricultural depression were even more severe on irrigated than on humid lands, as they were still in progress of development with credit resources strained to the limit. The most vulnerable creditor was the Federal Government. Relief in an acute situation, no doubt often well deserved, was found so easy to obtain, that a drive was then successfully made for more general relief. Although the relief granted may not have been justified in its entirety in all cases, the result has been such as to improve the morale of the settlers and to increase the revenue from these projects.

Much will be accomplished if the efforts by the Committee will aid in a new set-up that will remove the ever-present danger of a recurrence of these incidents.

Aided Settlement.—The Committee evidently has recognized that all is not well with the settlement of projects after water becomes available. It suggests stimulation by prevention of land speculation and recommends that financial aid should be a concern of the State and locality rather than the United States. The reference in the Committee report to aid extended by the Federal Land Bank is not pertinent, because the bank makes no loans on undeveloped farms.

In his discussion Mr. Mead describes the gap between completion of construction and the use of water. The turning point, however, is not reached until water has been used a sufficient length of time and to a degree corresponding with farm development to that point where the operating income exceeds the annual costs and leaves a surplus for retirement of debt.

In earlier years, this gap was bridged by moderate amounts of hardship extending over a few years. A far different situation now exists, due, in part, to: (*a*) Lower quality of new lands left for reclamation; (*b*) increase of farm equipment required; (*c*) changes in living standards; and (*d*) decline in farming profits compared with other industries.

The first two items are promptly reflected in an enormous increase in the cash capital, or its equivalent, required at the outset. The latter two have resulted in an extremely low net income even on fully developed lands and have forced out marginal lands. The margin of profit is wholly lost under conditions of incomplete development, and, if such conditions extend over a series of years, failure is inevitable. Lack of adequate capital is the most common cause for failure.

To meet the situation, the Interior Department has adopted and is practising certain measures of general application that require no investment, such as: (1) Prevention of entries on public lands by unqualified settlers; (2) curbing of land speculation; (3) compulsory district organization to stimulate development of private holdings; (4) dissemination of information to attract qualified settlers; and (5) assistance in the sale of private lands to "dirt" farmers.

These measures have contributed notably to improvement in settlement, but they do not wholly meet the need of adequate capital. The present requisite of $2 000 liquid capital, or its equivalent in stock and equipment, for the settler on public lands is generally too small to permit a proper start. Analyses indicate the need for fully twice that amount. Further increase in the required capital would rapidly reduce the ranks of eligibles and would introduce resentment by untold numbers who still cling to time-worn ideas of carving a homestead out of public lands with little more than their bare hands for assets. On developments that provide supplemental water, lands are largely privately owned, but there is at present no means to compel selection of buyers when large holdings are broken up, and, apparently, there is no firm legal foundation that will permit imposition of such requirement. It must be said, however, that local interests have had marked success in endeavoring to eliminate unworthy purchasers. While these measures have been effective, they are far from adequate. Unsuccessful efforts have been made to obtain settlement funds from Congress for a trial demonstration. Support for this proposal is not strong in or out of Congress, due, no doubt, to a general conviction that the Federal Government has done enough in constructing the irrigation works and because of a misgiving as to the possibility of maintaining a proper business relation where one party has so little at stake.

More than one effort has been made to obtain active and financial co-operation by States in settlement problems. In some cases no interest can be kindled, and nowhere has financial aid been forthcoming. In many quarters there is a decided tendency to consider the projects strictly a Government affair with all responsibility on the part of the State terminating when its representatives have brought about a start on construction.

The problem of settlement has not been solved; it warrants further serious study, because it will inevitably be a major factor in determining future reclamation policies.

Summing up the situation, the increasing difficulty of operations under the present plan and the growing criticism of the interest-waiver form of subsidy make it advisable to consider a change. The Committee's proposal of gratuitous construction of a part of the works has much merit and should be given further consideration. Settlement after completion of the engineering works is far from satisfactory and merits much more attention by those dealing with proposed changes in the Federal reclamation policy. State participation in construction and even more in settlement has been suggested, but all efforts to enlist the States in such work have been unsuccessful. If curtailment of development to reduce farm surplus is desirable and is adopted, it should be general and should be directed not only at irrigation activities, but impartially at irrigation, flood control, drainage, and all other activities throughout the country which either directly or indirectly result in increased production of those crops the overproduction of which is held to be responsible for depressed prices.

JOHN E. FIELD,[61] M. AM. SOC. C. E. (by letter).—As a member of the Committee, the writer considered the question referred to in Section (9) more from the viewpoint of its effect on the settler than on the market for farm products, the competition with existing farms, or the needs of the Nation for farm products. The question of first importance is: Can the settler succeed? If he succeeds in his enterprise and if, in consequence, some other farmer finds it necessary to give up farming, that is competition, the "survival of the fittest," or the evolution of the industry.

The increasing cost of transportation and distribution has tended to revive or sustain agriculture in the more densely populated areas, and such revival from a National viewpoint is more desirable than the reclamation of new and isolated areas, and will tend to supply any future deficiency in area cultivated.

The evolution in menus, the rapidly increasing area of crops grown under glass and canvas, the artificial stimuli of fertilizers and light rays, the development of better seeds and better methods, all indicate that, even without additional areas being brought under cultivation, there is no prospect of a food shortage for a long time to come.

This condition, supplemented by the tendency toward urban employment and habitation and the lack of demand for land for farming, justifies the principle stated in Section (9).

The movement of people in the United States has been westward since the founding of the Government; but in the decade, 1920-1930, there has been a general trend toward the South. With this movement southward of large numbers of people, although predominantly commercial and industrial, development will create a demand for land in that region. This local demand will be met with increased cultivable areas or more intensive cultivation.

---

[61] Denver, Colo.

The South is in strenuous competition with the West for new citizens, more capital, and more enterprises. One section of the West is the rival of another section of the West. The same is true of the South and of the Pacific Slope. Thus, reclamation of land is primarily a local, not a National, need; it should, therefore, be promoted and financed locally. The Federal Government, in so far as the Nation is benefited directly or indirectly, should aid these enterprises, but any other body, section, or region benefited should contribute in proportion to the benefits to be received. This principle is reflected in Sections (3) and (4) of the report.

"What's the hurry?" Many Governments have found it necessary to enter upon programs of large public undertakings involving the seeking of new territory and the founding of colonies, on account of the pressure of population, the necessity for more room, more work, and more revenue. Often, without such relief, the durations of government and even nations themselves have been threatened. Such conditions do not exist in the United States; nor will they for many years to come. It is all but certain, however, that, in time, this country will have like conditions to face and similar questions to answer. When that time comes it can be easily imagined that the people will deplore the too rapid growth of their country; they may wonder why the people of the present day were so desperately anxious to populate the country that aliens and undesirables were imported; that Americans had preferred to hand over their inheritance to strangers and to deprive their own descendants of opportunity. Coming back to the present, one may well ask, "What's the hurry?"

Every section has the right to grow and expand, and to do its utmost to develop its resources. This is a commendable characteristic, but the problem is local and not National, and it must be judged by its effect on posterity as well as on prosperity.

The explanatory statement by Mr. Lippincott is from the National rather than from the local viewpoint. As the membership of the Society is nationwide and as the subject is a "National" reclamation policy, the Committee strove to view the question Nationally, although of necessity the illustrations, discussions, and citations given in Committee meetings were local. Nevertheless, they covered quite one-half the area of the forty-eight States.

Throughout the report the principle is maintained that the States or local enterprise should be permitted to develop the natural resources, and that the Government should do only those things the States cannot do—such as the gathering of data over wide areas and long periods of time; but the report also holds that where the Government acts it should be in co-operation with the State—with the State's consent, and at its request. It is also stated in the report that projects not now feasible should not be undertaken at this time.

With reference to Section (1) of the report, it was suggested by some members of the Committee that the proposed program for the expenditure of $100 000 000 should be held in abeyance and subjected to the same rules of feasibility as new enterprises; this is expressed in Sections (10) and (11) of the report. It was agreed that the Government should limit the expenditures of this fund and "fulfill its assumed obligations" only.

In proposing a reclamation policy the desire to prevent foolish, excessive, or premature expenditures of money—whether Federal, State, or private—should be expressed. Some of the proposals for the expenditure of the $100 000 000 Reclamation Fund were foolish; others excessive in cost; and many were premature. One project likely to be built has all these three faults. If the States in which these projects are located had been required to share the financial responsibility, and the localities had been required to guarantee repayment, some, at least, of the projects never would have been suggested. In the future, localities and States should be required to guarantee, in part at least, the interest and repayment of principal, in accordance with the terms of a contract and schedule of repayment, before any project is undertaken.

The writer approves of applying a test question, "Can the project be made to pay?" It should be applied in such a way as to discourage inopportune construction; to deter those who could not or would not pay a reasonable interest on the advance made to them; to discourage speculation; to banish the thought that Government funds were free; and to impress on the settlers a sense of entire responsibility.

In analyzing the differences in attitude of the Government and of private enterprise as applied to the ownership of water, the writer believes that attachment of the water to the land is a mistake. In Colorado, titles to land and water are separate, and water may be transferred and its use extended to cover greater areas. This flexibility of use of water accounts in large measure for Colorado's marked progress in irrigation and her position as second of the arid States in the area of land irrigated. Similarly, the limitation of the area which one person could own on a Federal reclamation project was not entirely wise as it assumed that all settlers were equal in ability, enterprise, and resources; it assumed even that the number of children would be the same. A man of real executive and managerial ability would scarcely be content with an 80-acre tract in Colorado, Wyoming, or Nebraska, where only a living was in prospect, and where ultimate independence and the hope of providing advantages for his children were absent. The lack of experience and sufficient funds among the settlers was due, in part at least, to this limitation of the activities and prospects of the more experienced and able farmers.

State vs. Federal Control.—In Colorado the control and the responsibility for the distribution of water rests in appropriate State officials. Dual or joint control of the water by State and Nation would lead to confusion. The Enabling Acts of the States of the West give them the same standing as the original thirteen States. The public lands are held by the Government in trust for those who desire to use them. Except for this condition of trustee, all the land within its boundaries is owned by the State and has the same status as lands in private ownership. It is not possible, therefore, for the Government to assume the original ownership and control of the natural resource of water. That the Government has called State ownership into question is true; but the Western States are on equal footing with the original thirteen States. Can the Government, therefore, claim control and ownership of the water of New York, Connecticut, and Pennsylvania? This is a form of bureaucratic aggression which the original thirteen States may consider with profit to themselves.

The writer does not advocate nationalization of the rivers. He believes the province of the Government should be limited to investigation and research, to the gathering of data, and to aiding in the preparation of comprehensive plans for the ultimate maximum utilization of available water. He hesitates to advocate even the construction of large reservoirs unless it is distinctly provided that the control and distribution of the stored water remains in the State water officials.

The writer does not subscribe to Mr. Lippincott's explanatory statement that control of reservoirs should remain in the Federal Government if that means control of the water distribution. In Colorado there is an efficient organization for the distribution of the waters of the State. This control should, and of necessity will, remain in the State and under the State Engineer. A change from State to Federal or joint control can lead only to confusion because all canals and reservoirs now built are operated under State laws. Even the two Government projects (Uncompahgre and Grand Junction) receive their water (in accordance with the Reclamation Act) through the State water officials. Rather than submit to Federal distribution of water it would be better to forego Federal aid.

Considerable money has been spent in Colorado by the Government on these two projects. They are, or at least may yet be, advantageous to the State, but when one remembers the restrictions imposed on enterprises desiring to build irrigation works on the Rio Grande and North Platte Rivers, he is impressed with the fact that Colorado has gained nothing through Government aid; on the other hand, a question has been raised as to her right to administer and control the waters within her own boundaries. Interstate questions have been and can be settled by Court procedure or by compact. The writer consented to the recommendation in Section (2) that "the conservation of the water in the rivers and lakes of the country should be under public control," only because "public control" means generally State control. It means Federal control only where the States cannot function alone or by compact. This thought is stated in Section (3), as follows: "utilization of the waters should be undertaken by the States," or, as an alternative, "jointly by the United States and the States."

The thought lying back of Sections (1) and (10), as the writer sees it, is that it is better to fix definitely and irrevocably what the settler is to pay and when and how he is to pay it. Indefiniteness in this regard has led in the past to many of the evils now recognized. If the obligations are fixed and irrevocable before construction begins the unwise projects would be dropped; speculation largely eliminated; the incompetent deterred; and the experienced farmer encouraged to settle. It is a species of fraud not to tell a settler the cost of his land and water, and especially is it fraud to tell him what it will cost at the time he enters the contract and then increase it later without his consent. Similarly, it is a deception to obtain public funds under promise of repayment and then to pay no interest and to ask for the cancellation of part of the indebtedness. The contract, once made, should be carried out to the letter, just as contracts are carried out in the business world; some losses are inevitable in all business and some will be suffered in Federal reclamation, resulting in losses for the Government and bankruptcy for the individual. The one will charge it off to poor judgment in building the project and the other will charge it to his own foolishness, inexperience, or "hard luck." In addition, it is fraud to construct a project and induce settlement where the settler cannot achieve independence in his old age. It is a fraud to treat public money as "easy money."

Generally speaking, the cost of reclamation is in excess of what the settler can pay. Recognizing this condition the Government, the State, and the locality desiring to build a project should make up the difference under the assumption that either the direct or indirect benefits, immediate or remote, justify the necessary aid. The probable benefits to Nation, State, and locality can be determined in advance, and the settler, knowing that the amounts due from him must be paid, will put his mind and labor on his farm work rather than in devising ways and means to obtain remissions from the Government. Also, in the knowledge that he must pay for the operation, maintenance, and improvements, he will weigh the necessity before demanding expenditures.

Even if it were true that original capital costs the Government nothing, this would not excuse the undertaking of projects that were not feasible or for which the settler would require heavy subsidy in the form of no interest payments or the remission of the original cost.

The only basis for any other policy is the theory that every locality is equally entitled to its share in the contents of the "pork barrel." The fact that the Reclamation Fund is derived from the sale of lands in certain States is scarcely sufficient reason for undertaking projects that will not pay the costs; nor does the fact that the money comes from sales of lands or revenues from a certain State justify the theory that, "the money comes from our State, we want it spent here, it will give our people employment; we do not care whether the project is feasible or not."

To some this may be surprising, but as early as 1903 and as late as 1929, the writer has heard this sentiment expressed by eminent "native sons" advocating a project in their States.

A five-year period would be sufficient to develop a reclamation project, when the improvements are opportune and when a demand for land exists. However, where improvements are premature or where postponed payments encourage the holding of land without cultivation, twenty-five years for development would probably be required; in that time, however, prematureness would probably have disappeared.

The Colorado River Project with its Boulder Canyon Dam has been specifically mentioned as an enterprise which the Government alone can handle. This probably is true at present, but one can imagine either the seven Basin States, or the three lower ones, entering a partnership and compact to develop the basin. To declare (as some have done) that projects could not have been built without Government financing and Government contributions, is in the writer's judgment, an over-statement. There are projects which could not be built without Government aid. International questions might be such that the Government alone could function but, except that time is an essential element, the writer finds it difficult to imagine a project which a State, such as California, for instance, could not build. California is certainly equipped with engineers and scientists capable of undertaking the very largest projects. However, as long as Government financing will furnish the funds with the least expenditure of effort on the part of the States; as long as it will assume the risk and do the work, and will do these things in the shortest space of time, the line of least resistance will be followed. It is very easy to acquire the habit of being waited upon and relieved of work, worry, and responsibility. If hope of Government aid were not present one can imagine the States or even local enterprise and private capital undertaking the work—not at this time, perhaps, but at some future time.

It appears to the writer that the megalomania that is so characteristic of the United States is the real moving force back of most promotions, rather than any National or even State necessity. If there is a State necessity, then the State should assume the responsibility, and the part of the financial burdens needed to supplement the amounts that the settler can afford to pay. If such a rule prevailed the "log-rolling, pork-barrel" manifestations would abate, and loud shouts of "necessity" would sink to mere whispers.

Although the writer recognizes that extraordinary or supplemental aid is desirable from the settlers' point of view, and is often necessary on projects unwisely undertaken, he cannot subscribe to the universality of the need. In 1902, the Government examined the South Platte Project in Northeastern Colorado; it built the interstate North Platte Project less than 100 miles north. These projects were similar in elevation, soil, climate, markets, crops, and potential settlers. The supply of water in the South Platte was questionable, that of the North Platte, unquestionable. Several years later portions of the land in Colorado were provided with canals and storage reservoirs through the irrigation district plan. The districts have been successful. The cost of construction, including expenses of promotion, discount on bonds, interest during construction, and costs of investigations, was not greatly different from the costs on the North Platte Project, exclusive of drainage and costs of maintenance and operation. The difference is even less if the character and the permanence of the irrigation works built by the two agencies are taken into consideration. Nearly all the bonds of the districts have now (1930) been paid off. In the Colorado projects, the settlement was less rapid at first than on the Federal project, but the general character of the settlers was better in that they were more experienced and that the projects met local demands for land. Time, place, and cost corresponded with desire, demand, and necessity. A part of the indebtedness of the largest district was written off, in amount about equal to the promotion cost, and since then, in the knowledge of the futility of demanding further reductions or concessions, such demands have not been made and the district's credit is "A1." This illustrates, also, that land settlement can be taken care of locally. Had this been a Federal project less competent settlers would have rushed in; speculation based on the hope of avoiding repayment of costs would have been present; large maintenance, operation, and improvement costs would have been incurred on money advanced by the Government; and, probably, a demand for houses and leveling would have been made. As stated in Section (14) of the report, "Land settlement * * * or any other kind of assistance * * * should be treated as a local matter." If the locality and the State are unwilling to assume this duty and responsibility, construction should be postponed until demand and feasibility induce it to do so.

Referring to Sections (1) and (11), the contributions by the Federal Government should be based upon the estimated benefits to the country as a whole and nothing more. As a maximum, the contribution should not reduce the settler's obligations to less than what he can reasonably be expected to pay. The interest-payment rate should approximate the interest rate paid by the Government on its indebtedness. Such limitations and rates would undoubtedly leave obligations greater than the amount the settler could pay, and, in such cases, the project would be classed as premature and its construction postponed.

The proper desire of the U. S. Bureau of Reclamation has been to build projects so as to include all the lands possible. Often it was found that the earlier private enterprises took in only the choice areas, leaving the less desirable ones in such a condition that they could not be reclaimed feasibly. This suggests the popular expression, "they took the heart from the melon and left the rest." Carrying the simile further, the service in some cases undertook to "save the rind as well as the seed zone," and the results were unsatisfactory in that considerable areas were put under canal at great expense, but were later abandoned. A portion of the amounts written off by the Government were chargeable to this condition. Thus, reclamation went from the one extreme of private enterprise to the other by National expenditure. It is now proposed to construct a project in which only a very small percentage of the land to be irrigated can support a settler, even if both land and water, leveling, and fencing were given him at one-fourth the cost. The advisability of imposing the conditions of Section (10) is under such conditions apparent.

In Sections (8) and (9), the Committee's report indicates a preference for completing and bringing to perfection existing projects, and that great caution be exercised in attempting new projects. On existing projects much necessary expense has been met; facilities, such as roads, schools, and telephones, have been provided; small business centers have been built; and markets have been established. On a new project these facilities will have to be provided. It is obvious that the bringing of existing projects to a higher state of efficiency is the more economical and promises quicker results than undertaking new projects.

One of the primary purposes of the irrigation congresses, and of individuals proposing Federal aid, before the passage of the National Reclamation Act, was to furnish supplemental water supplies to existing projects and the extension of those projects. About the time the Reclamation Act was passed, the scope was enlarged to include construction of main canals. The writer does not remember that the construction of laterals, administrations of systems, building of drains, colonization, or aids to settlers, were proposed until after the passage of the Act, and some of these activities were not suggested for some years thereafter. It was when the activities undertaken were broadened in scope that needs for new contracts with increased cost over original estimates appeared. In the "pre-Act period," it was assumed that the settlers would be drawn from experienced communities and that they would be capable of self-government, would have initiative and ability, and would be able to cope with all the problems except that of the supply of water. The present tendency is to revert to the original conceptions of the irrigation congresses, and this thought is expressed in Section (8) of the report.

The writer believes that criticisms and distrust of Federal reclamation activities are decreasing, due, first, to the withdrawal by former Secretary Work, of the Department of the Interior, of restrictions of the rights of private enterprise to build reservoirs in Colorado on the Rio Grande and North Platte Rivers. While this reversal of previous rulings came too late to be of great benefit to that State, because the time when funds could be secured had passed, this saner and more equitable attitude of the Department of the Interior is appreciated. Second, the general conduct of the engineers of the Reclamation Service is winning ever greater popular approval. The present-day opposition and criticism is due to the attitude of the Government's legal advisers who claim that the title to water is in the hands of the Government and thus deny any weight to the article of the Enabling Act to the effect that all States are equal and are admitted on the same terms as the original thirteen States. Criticism is now more outspoken than formerly against the growth of bureaucracy, of which President Hoover, President Coolidge, and others have warned the Nation.

In the writer's experience the management of projects by the water users who pay the bills (Section 15), is more economical than that by either Government or corporations. In time, through experience, the water users become as efficient as and more conservative and less technical and theoretical than either.

Mr. Mead states it to be his conviction that "one of two things should be done: (1) Either provide aid in carrying out the second stage of reclamation; or (2) quit building canals to irrigate unimproved land." The writer agrees with this statement, but chooses the second alternative, because he believes that—except for local ambition, a megalomania, and the desire for local expenditures—there is no demand for such construction; and that, as indicated in Section (8) of the report, the construction of storage reservoirs for supplemental supplies and flood protection for existing projects meets the present need and also that for a very long time in the future. This need is aptly illustrated under the head of "Achievements of Federal Reclamation in Completing Private Development," in Mr. Mead's paper. The Salt River, the Weber Valley, and the Snake River are mentioned; the Uncompahgre Project, where a supplemental supply for existing canals was secured through a tunnel, and the North Platte Project, where many existing canals were supplied with a supplemental supply from Pathfinder Reservoir, are other examples. In the latter case a community of interest was created and conflicts of water rights were eliminated. On that project priority of right has now little meaning; but the matter of control and distribution remains a source of conflict and litigation in a three-cornered fight—in which Wyoming, Nebraska, and the Federal Government are concerned. Allocation of the water between the States and joint control by compact would solve the problem.

The explanatory statement by Mr. Mead is particularly worthy of study as it presents many facts pertinent to the question of reclamation by irrigation in general, and not to Federal reclamation alone. While the Committee's report was prepared by engineers, several of them were raised on irrigated farms; some now (1930) own and operate such farms; and all, probably, have had to deal with irrigation economics and with settlers.

It is probably impossible (outside the group of irrigation engineers) to secure a committee more familiar with all three phases that Mr. Mead mentions, namely, engineering, economics, and the settlement problem. Such being the case, it is vital that the subject be thoroughly discussed by engineers other than those of the Committee, and the fact that so many have entered the discussion and that ample time has been given for the discussions, is a matter for congratulations. While the discussions have been local in view application, taken in their entirety they have a National viewpoint. Out of them a correct conclusion will doubtless be reached that will be to the credit of the Engineering Profession in that it will demonstrate that engineers do not have as narrow an outlook as they are sometimes accused of having.

The subject discussed herein may be summarized as follows:

(*a*) There is no hurry; the Government should beware of camouflaged frauds in considering requests for Federal aid.

(*b*) Reclamation is at present a local and not a National question.

(*c*) Under present (1930) National conditions of market and crops the settler is likely not to succeed even with local, State, and Federal aid.

(*d*) To complete and perfect existing projects is desirable from a local viewpoint, although it is not necessary nationally.

(*e*) Willingness on the part of the State and locality to assume responsibility and a part of the cost should be the test of necessity and feasibility.

(*f*) Most projects that are feasible, desirable, and opportune can be built without Federal aid.

(*g*) The control and distribution of water should remain in the State.

COMMITTEE OF THE IRRIGATION DIVISION ON "A NATIONAL RECLAMATION POLICY" (by letter).—On November 1, 1926, the Irrigation Division appointed a committee composed of eleven Western irrigation engineers to consider and report upon "A National Reclamation Policy". After much correspondence two meetings of the Committee were held, and a unanimous report was finally adopted in San Francisco, Calif., in July, 1928. The report of the Committee was presented to the Division at San Diego, Calif., in October, 1928, and discussed in two statements by Elwood Mead, M. Am. Soc. C. E., and by the Chairman of the Committee, J. B. Lippincott, M. Am. Soc. C. E.

Twenty-four discussions have been presented during the two years after the publication of this report in September, 1928, by engineers of the United

States, Canada, Australia, and India. The Committee devoted more than a year's time to a consideration of its report which was unanimously signed before its presentation to the Irrigation Division. The Committee now has reviewed its original report, and the discussions, and does not find occasion to modify or amend its judgment of July, 1928.

President Hoover presented a letter, under date of August 21, 1929, to the Conference of Western Governors and Representatives, held in Salt Lake City, Utah, on August 27, 1929, relative to the control of public lands, minerals, and reclamation in the West. The parts of this letter that bear on the policy of Federal Reclamation are, as follows:

"It may be stated at once that our Western States have long since passed from their swaddling clothes and are to-day more competent to manage much of these affairs than is the Federal Government. Moreover, we must seek every opportunity to retard the expansion of Federal bureaucracy and to place our communities in control of their own destinies. The problems are in large degree administrative in character, both as they affect the Federal Government and the Government of the States."

As to the Reclamation Service, President Hoover continued:

"It seems to me that the vital questions here are to re-orient the direction of the Reclamation Service primarily to the storage of water and to simplify its administration.

* * * * * * * *

"The present Reclamation Act is based fundamentally on the reclamation of Government-owned lands. Possible areas available for reclamation have now passed almost wholly into private ownership and the use of the Reclamation Fund for further projects may be legally criticized owing to the fact that the land is no longer part of the public domain and circumlocution by voluntary agreements may not always be possible.

"Moreover, the application of the fund under the present organization results in very large Federal administrative activities within the States of a character which was never originally contemplated and which could be much better administered by the local State Governments themselves. In many ways it duplicates the State water administrations."

The position of President Hoover in connection with the policy of Federal Reclamation is of particular interest because of the many respects in which he presents similar opinions to those of the Committee.

The Committee calls attention to Principle (6) of its original report, which reads:

"(6) The output of power and of water at Federal or State works should be disposed of at wholesale and not at retail."

The intention of the Committee was to express its opposition to either the State or Federal Government entering into the distribution of either water or power to individual consumers thereof. Its belief was that the actual contact with such consumers should be through local organizations, such as districts, mutual water companies, or private corporations. The Committee recognizes that, as provided in the Reclamation Act, the control and distribution of the water are vested in the respective States.

The Committee endorses as a fundamental policy the proposal that the Federal Government should expend the money derived from the sale of public lands and the natural resources derived therefrom, in the arid land States, in the improvement and development of those States by the conservation and utilization of their water supplies. The Federal and State Governments should co-operate in the construction particularly of storage reservoirs for irrigation, flood control, and other water supply projects. The province of these reservoirs is to regulate the stream flow in such a manner as to make them of public benefit to all classes of water users irrespective of whether they are on Federal Reclamation Projects, or otherwise. This policy of development of the West is of great public importance not only to the Western States themselves, but to the Nation as a whole.

(Signed)

J. B. LIPPINCOTT, *Chairman,*
M. C. HINDERLIDER,
R. K. TIFFANY,
G. E. P. SMITH,
W. H. CODE,
JOHN E. FIELD,
GEORGE M. BACON,
JOSEPH A. ELLIOTT,
J. C. STEVENS,
BURTON SMITH (No expression of approval or disapproval received.)
C. E. GRUNSKY ("Without approval of the first sentence of the last paragraph because appropriations should be for specific aid and should not be made indirectly in unknown, uncertain amounts.")
Committee.

October 30, 1930.

MEMOIRS OF DECEASED MEMBERS

CLEMENS HERSCHEL, Past-President and Hon. M. Am. Soc. C. E.[1]

Died March 1, 1930

Clemens Herschel was born on March 23, 1842. He passed his boyhood in Davenport, Iowa, and, after studying with a tutor, he entered the Lawrence Scientific School of Harvard University at the early age of 16. Graduating in 1860 with distinction (*summa cum laude*), he took an advanced course in chemistry for one term, and then went to Europe to study French and prepare to enter the Ecole des Ponts et Chaussées, in Paris. As the number of foreigners was limited, and the quota was complete, he was unable to obtain admission, and went, as a consequence, to the Technical School at Karlsruhe, Germany, to complete his education. Long afterward, in 1925, the Karlsruhe Technical School bestowed upon him the honorary title of Doctor of Engineering.

In 1864, Mr. Herschel returned to the United States and opened an office as Consulting Engineer, in Boston, Mass., doing such work as came to hand. He was Engineer of the Albany Street Bridge, built by the City of Boston in 1867, and at one time was connected with the Boston Sewer Department. The variety of his work is indicated by a letter-head of 1871 which reads, "Civil Engineering in all its branches, Iron and other Bridges and Roofs, Hydraulic Engineering, Roads, River and Harbor Improvements, etc., etc." In 1872, he was appointed Superintendent of Streets of West Roxbury, Mass. (now part of Boston), and from 1881 to 1883 he was one of the three Railroad Commissioners of Massachusetts.

Mr. Herschel was Engineer of the Quinnipiac Drawbridge, New Haven, Conn., erected 1874-1878, and this led to the publication, in 1875, of his book on "Continuous Revolving Drawbridges." A bridge of minor importance, but which is of interest because it is still standing and readily accessible, is that across the Public Garden Pond in Boston. The design, prepared in cooperation with an architect named William G. Preston, was accepted as the result of a prize competition.

To one of Mr. Herschel's independence of thought, it was disappointing to note that "bridges and roofs" did not offer any future for a Consulting Engineer since their design was falling entirely into the hands of large bridge companies, where a designer became, as he expressed it, merely a "cog in a wheel". He had at one time worked under the late James B. Francis, Past-President and Hon. M. Am. Soc. C. E., a hydraulic engineer whom he greatly admired, and this, as well as the lack of a future in bridge work, naturally turned his attention especially to hydraulics.

In 1879, Mr. Herschel was appointed Hydraulic Engineer of the Holyoke (Mass.) Water Power Company. It should be remembered that power at

---

[1] Memoir prepared by J. Waldo Smith, Hon. M. Am. Soc. C. E., and William H. Burr, M. Am. Soc. C. E.

Holyoke was developed before the days of electrical transmission, and the Power Company not only controlled the dam, but had also to operate the canals which supplied water for power to the various mills. During the ten years that Mr. Herschel held this position he built and operated the Holyoke Testing Flume, then one of the few places in the country for measuring the efficiency of hydraulic turbines. All the turbines in use in the Holyoke mills were tested, and the discharge was determined for different gate-openings. Thus, every turbine became its own water meter, and by reading the gate-openings twice a day, an accurate record of the use of water for power purposes was obtained. This system was of great help in securing an equitable distribution of water among the different mills, supplied from the canals of the Holyoke Water Power Company, and in preventing waste.

This was not enough. The many paper mills used large quantities of wash water that did not pass through the turbines and was not measured. Mr. Herschel gazed on those wash-water pipes and pondered. He wondered how they could be made to tell how much water was passing through them. The Venturi meter was the answer, but it came too late to be used for measuring wash water. As he was wont to remark, the Venturi meter has been used for a great variety of purposes, but never for the purpose for which it was invented.

It had been observed by Venturi in 1791, that, if a liquid flows first through a converging and then through a diverging cone, the pressure at the narrow section where the two cones join is less than the initial pressure. To Venturi, the philosopher, this was merely an interesting phenomenon of Nature; to Herschel, it was a suggestion of a practical and much needed device, and, in 1887, he conducted a series of tests to determine whether the rate of flow could be calculated from this difference in pressure. Here, again, his training under Mr. Francis was of great assistance, as the latter had determined in tests on diverging cones, the small taper necessary for the down-stream cone of the meter, if the permanent or over-all loss of head should be a minimum.

The Holyoke tests were so satisfactory that the device was patented and was named the Venturi meter. Thus, Clemens Herschel became "the man who made Venturi famous". He was presented with the Rowland Prize by the Society for his paper describing his tests,[2] and was awarded the Elliott Cresson Gold Medal by the Franklin Institute for his invention.[3]

Another piece of important work at Holyoke was in practically rebuilding the wooden dam across the Connecticut River. This had become unsafe and threatened to give way, but when repaired and filled with gravel, it lasted as long as necessary, that is, until it was replaced after Mr. Herschel left Holyoke, by a stone dam built below it. In connection with the work on the dam Mr. Herschel added to the diversity of his accomplishments by going down in a diving suit, because he did not believe what the divers told him and he wanted to see for himself.

In 1889, Mr. Herschel was made Chief Engineer and Superintendent of the East Jersey Water Company, and gave up his position with the Holyoke

---

[2] *Transactions*, Am. Soc. C. E., Vol. XVII (July–December, 1887), p. 228.

[3] *Proceedings*, Franklin Inst., February, 1899.

Water Power Company. The East Jersey Water Company was organized to build works to provide a large additional water supply to Newark and other New Jersey communities. These works included dams and reservoirs on the Pequannock River, and a riveted, steel pipe line which was one of the first of that type to be built in the East. A contract was made on September 24, 1889, with the City of Newark, to build works to supply 50 000 000 gal. of water per day to that city, the works to be finished by May 1, 1892. Eventually, two pipe lines were laid and the capacity was increased to 80 000 000 gal.

It was on the works of the East Jersey Water Company that the Venturi meter had its first practical application. About thirteen meters were used and a daily record was kept of all water entering the pipes and of water delivered. From this beginning, the use of the Venturi meter has spread over the entire world, not only for water, but also for measuring gas and steam and other fluids. Noteworthy on account of their great size are the meters of the Catskill Aqueduct which measure the water consumption of New York City.

It was during Mr. Herschel's service with the East Jersey Water Company that he published, in 1897, "115 Hydraulic Experiments", on the flow of water in pipes and conduits. He left that Company in 1900, after the works for the City of Newark had been finished and turned over to the city.

Since 1884 Mr. Herschel had been Consulting Engineer of the largest power developments at Niagara Falls, including the work of the Cataract Construction Company and the Niagara Falls Power Company. In earlier developments at Niagara Falls, the head was subdivided or part wasted, and power was distributed by shafting. The ready-made turbines available at that time in the United States could not be used for the high heads and the large volumes of water available at Niagara Falls, and electrical transmission of power seemed desirable. To consider these questions, an International Commission of five members was appointed, Mr. Herschel being the only American member.

This work brought him into contact with European and especially with Swiss builders of hydraulic turbines, and the decision was reached to obtain from Europe, turbines built to order for the head and discharge available, and with a speed suitable for driving electric generators.

Turbines were first imported from Switzerland about 1904, but later the building of large power hydraulic turbines designed for predetermined conditions, was established in the United States, largely as a result of the experience of the Niagara Falls Power Company with Swiss turbines. Mr. Herschel was the first Manager of the Hydraulic Engineering Department of the Allis-Chalmers Company, but resigned when the Company's office was moved from New York to Milwaukee, Wis. With his intense love for Boston, he considered even New York as "too far West", and Milwaukee was unthinkable.

Mr. Herschel devised an application of the Venturi meter, which he called a "fall increaser", to increase the power of hydraulic turbines when low head was experienced in times of flood. This, however, was never put into practice, and conical draft-tubes, and other features of more recent turbine design, have rendered it unnecessary.

Mr. Herschel continued to act as Consulting Engineer with his office at 2 Wall Street, New York. He was one of a Commission of three engineers who examined the plans for the New York City Aqueduct in 1910 before they were approved by the Board of Estimate and Apportionment. In 1920, he read a paper before the American Society of Mechanical Engineers on "An Improved Form of Weir for Gaging in Open Channels", in which he described a method of reading pressures in a perforated pipe placed at the crest of the weir, and of calculating the flow from these readings.

He was a frequent contributor to the technical journals, and was always ready to champion the Venturi meter with vigor. Although he had written "A Farewell Word on the Venturi Meter", in April, 1929, at the age of 87, we find him back again on the firing line with a letter[4] to the Editor of *Engineering News-Record* written when confined to his home, and dated, Glen Ridge, N. J., August 28, 1929.

Among the obviously professional parts of Mr. Herschel's engineering career there is none more prominent than his contributions to what may be called Engineering Science. It was fundamental with him in seeking the solution of any important engineering problem to make a searching general analysis of all its features leading to results constituting a marked advance in engineering practice. His professional work from its beginning to its end exhibited such marked advances, as in his papers on drawbridges and other continuous trusses and on the Venturi meter, and in other cases of similar character. The same desire to give intellectual quality and finish to his professional work is gracefully shown in his translation of Frontinus on the water supply of Ancient Rome.

These characteristics of his practice as a civil engineer just at the time when Civil Engineering was taking its place as a profession in the United States made him one of its marked pioneers, especially in Hydraulic Engineering. His excellent educational equipment combined with a trained analytic mind was well timed to act upon the opportunities offered by the profession of Civil Engineering then coming into being. In fact, it is due to him and a few others associated with him that Hydraulic Engineering has been developed to its present status in this country.

His most remarkable literary achievement, and the culmination of years of study, was the publication, in 1899, of a translation from the Latin into English, of the "Two Books on the Water Supply of the City of Rome", by Sextus Julius Frontinus. Like many another, Mr. Herschel had forgotten the Latin learned in his youth, but, nothing daunted, he undertook the task with the help of a French and a German translation, and produced the first translation to appear in English.

Mr. Herschel enjoyed unusually good health until the last year of his life, and commuted daily from Glen Ridge, N. J., to his office in New York.

He was married, in 1869, to Grace D. Hobart, of Boston, who died in 1898; he was married a second time in 1910 to Jeanette B. Hunter, of Thompsonville, Conn. His children, all of whom survive, are Arthur H., Winslow H., and

---

[4] *Engineering News-Record,* September 12, 1929.

Clementine (Mrs. Hobart Rawson), by his first wife, and one son, Clemens, by his second wife.

Mr. Herschel was Treasurer of the Boston Society of Civil Engineers from 1874 to 1880, and was President of that Society in 1890–1891. At the time of his death he was Past-President and Honorary Member both of the American and of the Boston Society of Civil Engineers. He always loved Boston and the Boston Society, and directed in his will that his engineering books should be given to the Boston Society and to the Boston Public Library. His correspondence of many years has been given to the Engineering School of Harvard University.

In addition to the memberships mentioned, Mr. Herschel was a member of the Union Club of Boston, the Century and Engineers' Clubs, of New York City, and a Life Member of the Institution of Civil Engineers of London, England.

Mr. Herschel was elected a Member of the American Society of Civil Engineers on April 21, 1869, and an Honorary Member on June 19, 1922. He served as Director of the Society in 1891; as Vice-President in 1915 and 1916; and as President in 1916.

GEORGE SMEDLEY WEBSTER, Past-President, Am. Soc. C. E.[1]

DIED JANUARY 23, 1931

George Smedley Webster was born at "Stouton Farm", now McPherson Square, Philadelphia, Pa., on October 19, 1855, the son of John Hambleton and Lydia (Smedley) Webster. On his father's side he was a direct descendant of Nicholas Waln who came to America in the ship *Welcome* with William Penn, and on his mother's side, of George Smedley who settled in Middletown, Pa., about 1682. Both families were members of the Society of Friends. His father and grandfather were farmers, the latter having moved to "Stouton Farm" in 1805. Mr. Webster attended the Friends' Select School in Philadelphia, and later entered the University of Pennsylvania, from which he was graduated in 1875 with a Certificate of Proficiency in Civil Engineering. In 1909, he received the degree of Bachelor of Science and, in 1910, the honorary degree of Doctor of Science from this institution.

During the summer following his graduation, Mr. Webster was engaged as Transitman on surveys for a proposed gravity railroad around Fairmount Park, Philadelphia, and in the fall of that year became Assistant Engineer on the survey for the layout of the buildings and grounds for the Centennial Exposition held in Philadelphia the following year. This work being completed, he spent the fall of 1876 as an Assistant with the United States Coast and Geodetic Survey on primary triangulation in Eastern Pennsylvania.

In 1877, Mr. Webster entered the employ of the City of Philadelphia and continued to serve with the municipality for forty-four years. Starting as Draftsman in what was then the Department of Surveys, he became, successively, Inspector and Assistant Engineer. In 1880, he was made Surveyor and Regulator (Assistant Engineer) of the Tenth Survey District which comprised about one-third the total area of the city, having charge of the design and construction of sewerage systems, bridges, street systems, parks, and other engineering work for this large District. The position made him a member of the Board of Surveyors, a body charged with street and park planning and the approval of plans for major engineering works throughout the city. In the early part of 1892, he became Principal Assistant Engineer of the Bureau of Surveys, and, later that year, was promoted to the position of Acting Chief Engineer and Surveyor, and, in 1893, to that of Chief Engineer and Surveyor of the Bureau, which position he held until 1916 and, again, in 1920. During the interval from 1916 to 1920, he was Director of the Department of Wharves, Docks, and Ferries.

---

[1] Memoir prepared by Samuel Tobias Wagner, Henry H. Quimby, and Charles H. Stevens, Members, Am. Soc. C. E.

As Chief Engineer and Surveyor of the Bureau of Surveys, Department of Public Works, Mr. Webster was in charge of the extensive engineering work undertaken by the City. This covered all phases of municipal engineering, including the construction of bridges, elimination of grade crossings, sewerage, water purification, port development, and city planning. Any history of his accomplishments is practically a history of the development of Philadelphia during this period.

The more important bridge work undertaken by him includes the completion of the Walnut Street Bridge over the Schuylkill River, and the construction of the steel truss bridge, at the Falls of the Schuylkill, the electrically operated drawbridge over the Schuylkill River, at Grays Ferry, the bascule bridge over the Schuylkill River, at Passyunk Avenue, and the concrete arch bridge over Wissahickon Creek, at Walnut Lane. At the time of its construction, the Walnut Lane Bridge, with a main span of 233 ft., was the longest masonry arch in the United States and the longest concrete arch in the world. In 1915, Mr. Webster reported to the City Council on the feasibility of a bridge over the Delaware River connecting Philadelphia with Camden, N. J. The location and type of bridge recommended by him at this time were essentially the same as those finally adopted in 1921.

The elimination of grade crossings of the steam railroads was an important branch of the work of the Bureau of Surveys, which extended throughout Mr. Webster's entire term of office. This involved negotiations with the railroad officials, and, in conjunction with the railroad engineers, the design and construction of many highway and railroad bridges and structures. The more important projects include: The removal of twenty-nine crossings on the New York Division of the Pennsylvania Railroad by the construction of four-track and six-track bridges; the removal of thirty crossings on the Trenton Branch of the Pennsylvania Railroad by the construction of a two-track elevated railroad in Trenton Avenue, with bridges over cross streets; the removal of seventeen crossings on the main line of the Philadelphia and Reading Railroad by the construction of the Pennsylvania Avenue Subway, involving a four-track to six-track open subway, four-track tunnel, elevated railway, and nine bridges; the removal of twenty-eight crossings on the line of the Philadephia and Reading Railroad between Greene Street and Wayne Junction, by the construction of a four-track elevated structure; the removal of eight important crossings of from eleven to thirty tracks on the Richmond Branch of the Philadelphia and Reading Railroad, by the elevation of the line and the construction of bridges; and the preparation of plans for the removal of fifty-six crossings of the Pennsylvania and Baltimore and Ohio Railroads, in South Philadelphia.

More than 800 miles of sewers were constructed in the city during Mr. Webster's term as Chief Engineer of the Bureau of Surveys. He inaugurated and carried on research work for more than twenty years on the relation between storm-water run-off and rainfall in various parts of the city, which proved of great importance in the design of storm-water sewers. After visiting sewage treatment plants in Europe in 1908, he established a station for testing various methods of the treatment of sewage in Philadelphia, and, in the light of these experiments, he designed and built the first sewage treatment plant in the city. In 1914, he prepared, with two Assistant Engineers, a complete report on the collection and treatment of the sewage for the entire city, which was adopted and, in principle, is now being followed. The Governor of Pennsylvania appointed Mr. Webster a member of the State Sanitary Water Board, the duty of which is to administer the statutes relative to sewerage and the discharge of sewage into streams, and to pass upon all plans for sewers and sewage treatment works throughout the State.

The filtration of city water was inaugurated early in 1900, and Mr. Webster designed, built, and had charge of the operation of, the Spring Garden Water Testing Station. He also supervised the design of the Torresdale, Belmont, and Roxborough slow sand filter plants. From 1905 to 1906, in addition to his other duties, Mr. Webster was Acting Chief of the Bureau of Filtration.

One of his main interests was the establishment of the Municipal Testing Laboratory, one of the first of its kind in America. It was used at first to test the cement and concrete for city work, but its use was soon extended to include the testing of other materials entering into construction work and paving. Later, this was still further extended to include the testing of practically all materials purchased by contract for the city.

From 1892 until the establishment of the Department of Wharves, Docks, and Ferries in 1907, Mr. Webster was a member of the Board of Harbor Commissioners, having charge of the engineering work in connection with the improvement of the Port of Philadelphia. This included the widening of Delaware Avenue from Vine Street to South Street, the construction of municipal piers at Chestnut, Arch, and Race Streets, and the dredging of the Delaware River and Schuylkill River channels. When Mr. Webster was appointed Director of the Department of Wharves, Docks, and Ferries in 1916, he again had charge of the harbor improvement work. During the four years that he held this position the municipal piers at McKean, Kenilworth, and Cherry Streets, and numerous smaller piers were constructed.

From 1880 until 1916, Mr. Webster was a member of the Board of Surveyors and served as its President for twenty-four years. As mentioned previously, this Board was responsible for the planning of the city. Under his direction the first City Planning Division in America was established in the Bureau of Surveys, and some of the results of this can be seen in the development of the Parkway, Roosevelt Boulevard, Delaware Avenue, Lincoln Drive, Torresdale Avenue, and other important highways. At the time of his death, he was Vice-President and Chairman of the Technical Advisory Committee of the Regional Planning Federation of the Philadelphia Tri-State District, which is engaged in the preparation of plans for the co-ordinated development of the territory for thirty-five miles around Philadelphia.

While Chief Engineer of the Bureau of Surveys from 1892 to 1916, Mr. Webster was an *ex officio* member of the Fairmount Park Commission, a member of its Committee on Improvements, and, for many years, Chairman of the Committee on Land Purchases and Damages. During this period he was also a member of the Board of Highway Supervisors, and, for more than twenty years, was its Presiding Officer. When the Transit Commission was formed in 1912 to make a study of the transit situation, Mr. Webster was appointed Acting Transit Commissioner during the illness of Transit Commissioner A. Merritt Taylor. The preliminary studies made and the work done by the Commission led to the formation of the Department of City Transit.

Mr. Webster resigned from the Bureau of Surveys in 1920 when selected by the Delaware River Bridge Joint Commission, together with Ralph Modjeski and Lawrence A. Ball, Members, Am. Soc. C. E., as a member of the Board of Engineers to consider sites and prepare plans and estimates for the Delaware River Bridge. The report of the Board was submitted to the Committee the following year, and the Board of Engineers was retained for the construction of the bridge. At the time of its completion in 1926, the Delaware River Bridge was the longest suspension bridge ever built, the length of the main span being 1 750 ft., and the total length, including approaches, 9 570 ft. In 1928, Mr. Webster was engaged as a Consulting Engineer, with Mr. Modjeski and C. E. Chase, M. Am. Soc. C. E., on the Henry Avenue Bridge, over Wissahickon Creek.

President Roosevelt appointed Mr. Webster a member of the Advisory Board on Structural Materials. On nomination by the American Society of Civil Engineers, he also became a member of the Engineering Division of the National Research Council.

Mr. Webster was especially interested in the American Society for Testing Materials, of which he was elected a member in 1900, and of which he was Vice-President in 1918 and 1919, and President in 1920. In addition, he served on various committees, as follows: The Committee on Cement (C-1), from 1903 to 1931, being Vice-Chairman from 1903 to 1915; the Committee on Reinforced Concrete (C-2), from 1904 to 1931; and the Committee on Clay and Concrete Sewer Pipe (C-4), from 1910 to 1922. He also presented a paper before that Society in 1902 on the "Advantages of Uniformity in

Specifications for Cement and Methods of Testing." From 1906 until its activities were suspended at the beginning of the World War, Mr. Webster was also a member of the International Association for Testing Materials.

He was the first to be elected to membership in the Engineers Club of Philadelphia after its organization in 1878. He served as Director of the Club in 1891 and as President in 1895, and was always actively interested in it. From time to time, he presented papers before the Club covering municipal projects under his charge, the most notable being the report of his investigations of sewage works in Europe.

He was a member of the Engineering Alumni Society of the University of Pennsylvania and its President in 1911; the Society of Municipal Engineers of the City of Philadelphia, of which he served as President in 1914; Past-President of the Sanitary Engineering Section of the American Public Health Association; member of the Franklin Institute; National Conference on City Planning; Chamber of Commerce of Philadelphia; Union League of Philadelphia; Honorary Member of the Rotary Club; Honorary Member of the T Square Club; Honorary Member of the City Parks Association; member of the Fairmount Park Art Association; Pennsylvania Forestry Association; Historical Society of Pennsylvania; Historical Society of Frankford; Frankford-Torresdale Country Club; and of the Board of Managers of the Pocono Manor Association. He was a member of the Society of Friends and, at the time of his death, was President of the Board of Managers of the Friends' Hospital in Philadelphia.

The following papers were presented to the Society from time to time by Mr. Webster: Final Report of the Special Committee on Uniform Tests of Cement[2]; "History of the Pennsylvania Avenue Subway, Philadelphia, and Sewer Construction Connected Therewith"[3] (joint paper with Samuel Tobias Wagner, M. Am. Soc. C. E.); "Specifications and Methods of Tests for Portland Cement"[4]; "The Pennsylvania Avenue Subway and Tunnel, Philadelphia, Pa."[5] (joint paper with Mr. Wagner); and "Walnut Lane Bridge, Philadelphia"[6] (joint paper with Henry H. Quimby, M. Am. Soc. C. E.).

Mr. Webster was a man who commanded the admiration of all who came in contact with him. He devoted his entire life to the planning and development of public works for the betterment of his native city and in the service of its citizens. His knowledge of conditions in Philadelphia and his breadth of vision in planning for its future growth were outstanding. Serving under eleven Mayors in succession, he was retained in important executive positions until he resigned from the City service on his appointment to the Board of Engineers of the Delaware River Bridge Joint Commission. An indefatigable worker, he demanded similar service from those who worked for him. He watched after the interests of the City as if they were his own. Straightforward, upright, positive in his convictions, a real diplomat in the public service, he gave to the citizens of Philadelphia an engineering service that will be hard to equal.

---

[2] *Transactions*, Am. Soc. C. E., Vol. LXXV (December, 1912), p. 665.

[3] *Loc. cit.*, Vol. XLIV (December, 1900), p. 1.

[4] *Loc. cit.*, Vol. LXXXII (December, 1918), p. 166.

[5] *Loc. cit.*, Vol. XLVIII (August, 1902), p. 470.

[6] *Loc. cit.*, Vol. LXV (December, 1909), p. 423.

He was married on December 5, 1883, to Mary H. Anderson, of Philadelphia, who, with their son, Maurice A. Webster, survives him.

Mr. Webster was elected an Associate Member of the American Society of Civil Engineers on September 7, 1892, and a Member on October 4, 1893. He served as a Director of the Society from 1904 to 1906; as Vice-President in 1917 and 1918; as President in 1920; and on the Board of Direction as Past-President from 1921 to 1922. He also served as President of the Philadelphia Association of Members (now the Philadelphia Section), of the American Society of Civil Engineers.

GEORGE DAVIDSON, Hon. M. Am. Soc. C. E.[1]

Died December 1, 1911

George Davidson was born at Nottingham, England, on May 9, 1825. He came to the United States with his parents in 1832, and his early education was obtained in the Central High School, at Philadelphia, Pa. He was employed as Magnetic Observer at Girard College, in Philadelphia, from 1843 to 1845, and on June 1, 1845, he entered the service of the United States Coast and Geodetic Survey, as a Computer and Secretary to Superintendent Bache. He was appointed an Aid in 1846, and during the last twenty-seven years of his service was in charge of all Coast Survey operations on the Pacific Coast of the United States, until his resignation on June 30, 1895.

From 1846 to 1848, while serving as an Aid, Mr. Davidson was a member of survey parties engaged in making astronomical and meteorological observations, and, from that time, his interest in astronomy never flagged.

The necessity for an accurate knowledge of the Pacific Coast in the interests of navigation was recognized by the Federal authorities in Washington, D. C., and, in the summer of 1850, Mr. Davidson was appointed Chief of a party for making the needed surveys. The first requirements were the determination of the geographical positions of the prominent capes and headlands, the hydrographic surveys of the principal harbors, and the topographic surveys of the shore line adjacent to the harbors. Mr. Davidson's first observing station on the Pacific Coast was at Point Conception, California, one of the most prominent and dangerous on the coast. Here, he determined the latitude and longitude of the station, the variation and dip of the magnetic needle, and also reported on the best location for a lighthouse in that neighborhood. During the next four years, stations were occupied in California, successively, at Monterey, San Diego, Cape Disappointment, Port Orford, Neah Bay, the Presidio of San Francisco, and at approximately twenty minor points.

At that time only two methods were available for obtaining longitude, one by moon culminations and the other by the observed times of occultations of stars by the moon. Both these methods were used by Mr. Davidson. Travel along the coast in those days was by sailing vessels, and dangerous landings through the surf were often unavoidable. Sometimes there was danger from hostile natives, as at Neah Bay, where the observations were made behind breastworks, with men fully armed and ready to shoot.

---

[1] Memoir prepared by Fremont Morse, Esq., Berkeley, Calif., and presented by Paul C. Whitney, M. Am. Soc. C. E.

A report by Mr. Davidson, entitled "Directory for the Pacific Coast", giving the results of his accumulated observations, was issued in 1855 as a guide to mariners. In the Coast Survey report for 1858, the Superintendent refers to this Directory as follows:

"This useful work has been compiled by George Davidson, Assistant in the Coast Survey, from Coast Survey and other authentic data, embodying also the results of his own experience on the coast, and topographical parties since the commencement of the survey in 1850. Having thoroughly and peculiarly identified himself with the survey of the western coast from its beginning, and having occasion himself to know the necessities, facilities and dangers of its navigation, he has been in a position to prepare a particularly valuable directory for the use of mariners and navigators."

This Directory was later known as the "Coast Pilot of California, Oregon, and Washington," and has passed through many revised editions.

Early in 1867, Mr. Davidson was appointed as Engineer of the exploration party dispatched by the Government to the Isthmus of Darien for the purpose of finding a practicable location for an Isthmian Canal. Immediately following the purchase of Alaska from Russia, he was ordered to make a preliminary geographical survey of that Territory, and, in 1868, the results of these accumulated observations were published in the "Coast Pilot of Alaska", of which he was the author.

The partial eclipse of May 26, 1854, was observed by Mr. Davidson at Humboldt Bay, California, chiefly for times of contact of solar and lunar images. He observed the occultations of twenty-two stars of the Pleiades group on March 1, 1857, and, to strengthen the longitude determination of the San Francisco Station, he observed at San Francisco the partial solar eclipse of March 25, 1857. He observed occultations of the Pleiades on March 25, 1860, at Ross Mountain, and the partial solar eclipse of July 18, 1860, on Sonoma Mountain.

Because of his intensive work on the astronomical foundation of the Pacific Coast surveys, any weak point in the instruments used was brought to his attention, and many improvements were suggested because of his inventive genius. A description of his meridian instrument for determining time, longitude, and latitude, which is used extensively by the Survey throughout the United States, was published in the report for 1867.

The construction of the Union and Central Pacific Railways, and the establishment of telegraphic connection between the Atlantic and Pacific Coasts enabled the longitudinal determinations of the West Coast to be placed on an accurate basis for the first time. The Pacific end of the longitudinal comparison with Harvard College Observatory was conducted by Mr. Davidson in a temporary observatory set up for the purpose in Washington Square, San Francisco. Telegraphic signals were exchanged with Cambridge, Mass., on twelve nights between February 15 and April 4, 1869, through the courtesy of the Western Union Telegraph Company. Mr. Davidson had already taken part in the transatlantic longitudinal determination in 1866, conducted by means of the new Atlantic cable.

The total solar eclipse of August 7, 1869, was observed by Mr. Davidson on the Chilkat River, Alaska, and, in 1870, he recorded the flight of 556 meteors at the San Buena Ventura Station, and made a special report on the subject.

In 1873, his party surveyed and permanently marked the eastern boundary line of California, north of Lake Tahoe, fixing the position of the 120th meridian west of Greenwich, and from 1893 to 1899, the diagonal boundary line from Lake Tahoe southeasterly to the Colorado River, was run.

The transit of Venus had been observed in 1769 by M. Chappe de l'Auteroche, at San José del Cabo, in Lower California, but this observer had not determined, or at least had not published, the exact position of his observing station. On account of the interest attached to the transits of Venus which occurred in 1874 and 1882, Mr. Davidson, in 1873, searched for the exact spot occupied by de l'Auteroche. Although no evidence of the latter's occupation remained, the description of the station enabled Mr. Davidson to locate it within a space of 20 sq. ft., and the co-ordinates of the observing station were accordingly determined.

Mr. Davidson was appointed by the United States Government Commission to observe the transit of Venus, in Japan, on December 8, 1874. On that occasion he determined the difference of longitude between Nagasaki and Tokyo, telegraphically, at his own expense.

He was a Delegate from the Federal Government to the Paris Exposition of 1878, with instructions to examine and report upon the instruments applicable to geodesy and astronomy. He was elected President of the Jury on Machinery by the French and foreign jurors. This Jury examined 3 800 pieces of machinery and awarded 850 prizes. Mr. Davidson was awarded a diploma and a medal for this service by the French Government.

The total solar eclipse of January 11, 1880, was carefully observed from the summit of Santa Lucia Mountain, California, by a Coast Survey party under his direction. He also directed the United States Government party for the observations of the transit of Venus, at Cerro Roblado, New Mexico, in 1882.

In the late Eighties, the question of the variation of terrestrial latitudes was prominent, and observations at widely separated stations were urgently requested. Mr. Davidson undertook the observation of latitude pairs, by the Tallcot-Horrebow method, at his observatory in San Francisco, between May, 1891, and August, 1893. An additional series was made by him in 1893 and 1894. The results confirmed those secured at European, Atlantic Coast, and Hawaiian stations, establishing the fact that the astronomical latitudes of points on the surface of the earth are constantly changing by minute amounts.

Mr. Davidson's programs of observation, whether for the determination of latitude, time, azimuth, magnetic declination and dip, refraction constants, or for research in theoretical astronomy on his own account, were characterized by the great number of observations planned for and acquired as well as by the observance of precautions for ensuring that the individual observations be as accurate as possible.

It is a remarkable fact that the first investigating astronomical observatory planned for the western half of the United States—the Lick Observatory—was on a large scale for its time. The first research observatory completed, however, was that of Mr. Davidson. This was established in Lafayette Park, San Francisco, about 1879. It contained a 6.4-in. equatorial telescope (by Mr. Alvan Clark) and auxiliary apparatus. Here, Mr. Davidson made drawings of the principal planets and observed star occultations and comets. The instruments were transported to stations occupied for the various eclipse observations.

Mr. Davidson was one of the three Commissioners sent by the United States Government in 1875 to East Asia, India, Egypt, and Europe, to report on the methods and state of irrigation in those countries. He was also a delegate from the United States to the Conference of the International Geodetic Association held in Paris, France, in 1889, and was commissioned to bring to Washington the international prototypes of the stardard meter and kilogramme.

His life was devoted with untiring energy and complete unselfishness to the interests of science on the Pacific Coast. It is no exaggeration to say that during sixty years his name was more familiar to the scientifically inclined of that region than that of any other resident. His services were not confined to the requirements of the Coast Survey work, and his efficient help was always available for the advancement of any department of science.

Professor Davidson was an authority on the voyages of the early explorers of the Pacific Coast and on its early history. The subject of astronomy, theoretical and practical, seemed to make a special appeal to him. He was well versed in the geodetic applications of astronomy, and the reports of the Coast Survey show that he never failed to grasp opportunities for making observations promising to be useful in any department of theoretical astronomy. The list of his published contributions contains many titles relating to eclipses, occultations, meteors, comets, and to atmospheric conditions affecting astronomical observations. His influence with James Lick, while Mr. Lick was formulating plans for erecting the most powerful telescope in existence, was wise and potent, and perhaps even vital to a practical solution of Mr. Lick's problem. In fact, it is entirely possible that without Professor Davidson's guidance at a critical period the plans for the construction of a useful telescope and observatory might have come to grief.

He was President of the California Academy of Sciences for sixteen years, and of the Pacific Geographical Society for thirty years. At various times and during periods of different length, he served on the Federal Commission for Irrigation, in California, the United States Advisory Harbor Commission of San Francisco, the Mississippi River Commission, and the United States Assay Commission, as Regent of the University of California, and, during the last ten years of his life, as Professor of Geography in the University of California.

The high merit of Professor Davidson's work was recognized by the conferring of honorary degrees and election to membership in many learned societies. He received the degree of Master of Arts from the Central High School of Philadelphia, in 1850; the honorary degree of Doctor of Philosophy from Santa Clara College, California, in 1876; the honorary degree of Doctor of Sciences from the University of Pennsylvania, in 1889; and the degree of Doctor of Laws from the University of California, in 1910. He was made a Life Member of the Academy of Natural Science of Philadelphia, in 1885. He was elected a member of the American Philosophical Society in 1886; a member of the National Academy of Sciences in 1874; a Fellow of the American Association for the Advancement of Science in 1880; an Associate Member of the American Academy of Arts and Sciences in 1887; a Corresponding Member of the Bureau of Longitude of France in 1894; and a Foreign Correspondent of the Academy of Sciences of France in 1901. He held Honorary Membership in the American Geographical Society, of New York, the Royal Geographical Society of London; the Swedish Society of Anthropology and Geography; and the Berlin Geographical Society. He was a member of the Seismological Society of America (President, in 1907), the California Academy of Sciences, the Pacific Geographical Society, and the Washington Academy of Sciences. He was created a Knight of the Royal Order of St. Olaf of Norway in 1907, and he received the Charles P. Daly Medal of the American Geographical Society in 1898.

On October 8, 1858, he was married to Elinor Fauntleroy, of Virginia, and made his home in San Francisco until his death on December 1, 1911. He is survived by a son, Thomas Davidson, and a daughter, Elinor Davidson, both of San Francisco.

Fremont Morse, Hydrographic and Geodetic Engineer (*Retired*), U. S. Coast and Geodetic Survey, one of the few officers still living (1931), who served with Professor Davidson, writes of him as follows:

"He was a man of strong likes and dislikes. As a chief of party, he was the finest I have ever served under. If he had confidence in you and knew you could handle any situation that might arise, he would never give you any detailed instructions when assigning you to perform a piece of work; but if at any time you felt you needed some help, he was never too busy to interrupt anything he might have in hand and would willingly give you the benefit of his great experience."

Professor Davidson was elected an Honorary Member of the American Society of Civil Engineers on May 5, 1897.

GRENVILLE MELLEN DODGE, Hon. M. Am. Soc. C. E.[1]

DIED JANUARY 3, 1916

Grenville Mellen Dodge was born on April 12, 1831, at Danvers, Mass. His parents were Sylvanus and Julia Phillips Dodge, people of gentle birth whose progenitors had come from England early in the Seventeenth Century, and had settled in the vicinity of Rowley, near Boston, Mass. His parents were poor, his father in later life becoming Postmaster of Danvers.

---

[1] Memoir prepared by George H. Pegram, Past-President, Am. Soc. C. E.

When the boy was thirteen years of age he went to work on the large farm of Mrs. Lander on the outskirts of Danvers, where he also assisted in his first surveying work. Through contact in this work with the son, Frederick Lander, the boy was fired with the ambition to prepare himself for college. He matriculated at Norwich University, Northfield, Vt., in the autumn of 1848, from which he was graduated in 1851, with the degree of Military and Civil Engineer. In 1892 he was further honored by the degrees of Master of Arts, Master of Military Science, and Doctor of Laws from Norwich University, and, later, the degree of Doctor of Laws which was conferred on him by Cornell College, Iowa, in 1904.

Immediately after his graduation Mr. Dodge went to Illinois, where he entered railroad work. In 1852, he served as Assistant Engineer on the Illinois Central Railroad, on the survey of the line from LaSalle to Dixon, Ill. In 1853, Chief Engineer Peter A. Dey, of the Chicago and Rock Island Railway Company, gave him charge, as Principal Assistant Engineer, of a survey of the Mississippi and Missouri River Railroad (now a part of the Chicago, Rock Island and Pacific Railway System), across Iowa, from Davenport to Council Bluffs, and he determined the latter point as the proper crossing of the Missouri River for a transcontinental line.

At various times between 1853 and 1861 Mr. Dodge made explorations west of the Missouri River and in the Rocky Mountains for the purpose of finding the best route for a transcontinental railroad.

He was employed on the construction of the Mississippi and Missouri Railroad until 1857, when with the cessation of railroad activities, he became a member of the firm of Baldwin, Pegram and Company, engaged in banking and mercantile interests, and freighting across the plains to Denver, Colo. He continued with this firm until the opening of the Civil War in 1861.

Mr. Dodge had always been interested in military training and was Captain of the Council Bluffs Guard when he was offered an appointment as Captain in the Regular Army, which he declined in order to accept a commission as Colonel in the 4th Iowa Infantry, which entered service at Rolla, Mo., on August 24, 1861. He was assigned to a post at Rolla on October 9, and commanded expeditions to Houston and Salem, Mo., on November 1. The enemy was defeated in both engagements.

Some of Colonel Dodge's further engagements were as follows: Assigned to command of 1st Brigade, 4th Division, Army of the Southwest, January 21, 1862; advanced to and occupied Springfield, February 13; in engagement at Sugar Creek, Mo., February 17, and Cane Creek, Mo., February 20; defeated Gates' command at Blackburn's Mills, February 27; Battle of Pea Ridge, March 6 to 8, during which he was wounded in the right side; commissioned Brigadier General of Volunteers, March 31; reported to Major-General Halleck at Corinth, Miss., June 6, and ordered by him to report to Major-General Quimby commanding the District of Columbus, Kentucky, to re-build the

Mobile and Ohio Railroad; assigned to the command of the Central Division, Army of Tennessee, with headquarters at Trenton, Tenn., June 28; organized and mustered into service, 1st Tennessee Cavalry; finished re-building Mobile and Ohio Railroad, in August, and built stockades and earthworks at all its important bridges and stations.

During the time General Dodge was in command at Trenton, the captures of Dyersburg, Huntington, and O'Brien, Tenn., were made, and Villepigue was defeated on the Hatchie River.

On September 29, 1862, by order of Maj.-Gen. U. S. Grant, General Dodge was assigned to the command of the District of Columbus, Kentucky. Extensive fortifications and important works in and around Corinth were finished while he held that command.

On December 1, 1862, a combined movement was made from Holly Springs and Corinth during which the troops of General Dodge captured Tupelo and Okolona, Miss., defeating the enemy and taking the stores at those places. During the same month, by order of General Grant, he moved to Lexington and Spring Creek, Tenn., driving Forrest across the Tennessee River. In February, 1863, he attacked Van Dorn's column at Tuscumbia, Ala., and captured the place with its stores, artillery, etc.

In April, 1863, while in command of the 2d part of the 5th and a part of the Cavalry Divisions of the 16th Army Corps, he conducted an expedition in Northern Alabama, defeating the forces of the enemy in the Tuscumbia Valley. During the movement, the fights at Bear Creek, Cherokee, Burton Station, Leighton, and Town Creek occurred. Immense quantities of stores for Bragg's Army were captured and destroyed. During the same month, Chalmers and Ruggles were attacked and defeated at Tupelo. In June, General Dodge crossed the Tennessee River at Savannah, Tenn., and moving into Van Dorn's rear, captured the Town of Florence, Ala., defeating its garrison. On June 19, he attacked Ferguson's command on Big Muddy and stopped the raid on the Memphis and Charlestown Railroad. The fighting was severe and the loss considerable. On July 5, after the capture of Vicksburg, Miss., General Dodge was recommended by General Grant as first for promotion to the rank of Major-General. On July 8, he was assigned to the command of the left wing of the 16th Army Corps, with headquarters at Corinth. On the 14th of that month he defeated a part of Roddy's command at Jackson, Tenn., and captured many prisoners. On August 15, he made a raid on Granada, Miss., captured a large number of locomotives, cars, railroad stock, stores, etc., and defeated the enemy at Water Valley and Granada.

While at Corinth, General Dodge organized and mustered into service the 1st Alabama Cavalry, the 1st Alabama Colored Infantry, and several companies of colored troops for siege artillery. On October 30, his command was attached to that of Major-General Sherman, then moving into Middle Tennessee.

During November and December, 1863, he rebuilt the Nashville and Decatur Railroad, also pontoon bridges across the Tennessee River at Decatur, Ala., and the Duck and Elk Rivers, and constructed good and substantial earthworks and stockades at all the important bridges and points. During this time the command had several engagements with the enemy, and captured and fortified Decatur.

In January, 1864, General Dodge organized and mustered into service the 2d and 3d Regiments of the Alabama Colored Infantry. In May, he was in command of the 16th Army Corps in the field in the Atlanta Campaign, and took part in the battles and engagements at Ship's Gap, on May 6; Snake Creek, on May 9; Resaca, on May 11; Estonula River, on May 12; Kenesaw Mountain, Roswell, Dallas, and Decatur, on July 21; Atlanta on July 22; and Ezra Church on July 28. On June 7, 1864, he was commissioned Major-General.

He was wounded in the head on August 19, and, on October 14, was ordered to City Point, Va., to visit General Grant. On November 3, General Dodge was assigned to the command of the District of Vicksburg and of the left wing of the 16th Army Corps. On December 2, he was assigned to the command of the Department and Army of Missouri, and on the 9th of that month was commissioned Major-General of the Missouri State Militia.

On January 30, 1865, the Department of Kansas was added to the Department of Missouri. Gen. Jefferson C. Thompson surrendered the Confederate forces in Arkansas. During January, February, and March, General Dodge made Indian campaigns on the plains, opening up the stage lines, and rebuilding telegraph lines destroyed by the Indians.

During 1865 and 1866, in his Indian campaigns, extending from the Arkansas River on the south to the Yellowstone on the north, many severe battles were fought. Treaties of peace were made with the Comanches, Apaches, Southern Cheyennes, and other Southern tribes, and a Council with the Northern Cheyennes, Arapahoes, and Sioux at Fort Laramie established the basis for a treaty. For service in this campaign, General Dodge received the thanks of the Legislature of Iowa.

In his "Memoirs" General Grant states, in part: "General Dodge besides being a most capable soldier, was an experienced railroad builder," and, referring to the re-construction of the railway between Nashville and Stevenson, he says further:

"General Dodge had the work assigned to him finished within forty days after receiving orders. The number of bridges to re-build was one hundred and eighty-two, many of them over deep and wide chasms. The length of the road repaired was one hundred and two miles."

On May 30, 1866, General Dodge resigned from the Army and immediately took charge, as Chief Engineer, of the construction of the Union Pacific Railroad. The transportation of supplies was one of the serious difficulties. No railroad had yet reached Council Bluffs, the initial point, and supplies had to be shipped from St. Louis and St. Joseph, Mo., by boats, then across the plains through a hostile Indian country, by teams.

The road was completed to a connection with the Central Pacific at Promontory Point, Utah, on May 10, 1869, 556 miles having been built in the last year—the most rapid railroad construction of record.

General Dodge realized the importance of intensive preliminary study of the ground. In the location of the Union Pacific Railroad, 25 000 miles of reconnaissance were made, and 15 000 miles of instrument lines were run. As a result the Union Pacific was originally so well located that subsequent improvements have shortened the road but thirty miles.

General Dodge was America's greatest railroad builder. He constructed the Union Pacific from Council Bluffs, Iowa, to Ogden, Utah; the Texas and Pacific from New Orleans, La., to El Paso, Tex.; the Fort Worth and Denver City, from Fort Worth, Tex., to Trinidad, Colo.; the Missouri, Kansas and Texas, from Fort Worth to Taylor, Tex.; the International Railroad of Texas from San Antonio to Laredo, Tex.; and the Des Moines and Northern (now part of the Chicago, Milwaukee and St. Paul Railway System). He was engaged as well in an advisory capacity on the construction of other lines. As President of the Mexican and Southern Railroad, he supervised the surveys from the City of Mexico through the State of Oaxaca, to the Pacific Ocean.

General Dodge at times was President of the following Railroad Companies: St. Louis, Des Moines, and Northern Railway; Des Moines Union Railway; Denver, Texas, and Fort Worth Railroad; Des Moines and Northern Railroad; and Union Pacific, Denver, and Gulf Railway. He was a Director of the Union Pacific Railway Company, and of the Wichita Valley Railway and Iron Steamboat Company.

General Dodge also had served, either as President, Director, or Chief Engineer, of the following companies: The Pacific Railway Improvement Company; the American Railway Improvement Company; the International Railway Improvement Company; the Texas and Colorado Railway Construction Company; the Oriental Construction Company; and the Colorado and Texas Construction Company, and the Western Industrial Company.

On the death of General Sherman in 1892, General Dodge became President of the Society of the Army of the Tennessee, which office he held until his death. He was Vice-President of the Grant Monument Association and Grand Marshal of the parade at the dedication of Grant's Tomb in New York, N. Y. He was also one of the organizers of the Loyal Legion and served as its Commander-in-Chief from 1907 to 1909.

In 1886-88, he represented the First Iowa District in Congress, declining re-nomination on account of the demands of his railroad work. In 1898, he was appointed by the President as Chairman of the Commission, known as the "Dodge Commission," to inquire into the management of the war with Spain.

In addition to his frequent contributions to periodicals, General Dodge published "How We Built the Union Pacific Railway, and Other Railroad Papers and Addresses" (1911), and, also, "The Battle of Atlanta."

He was married on May 29, 1854, to Annie Brown, of Peru, Ill., who, with their three children, Lettie (Mrs. Robert E. Montgomery), Ella (Mrs. Frank S. Pusey), and Annie, survives him.

General Dodge was patient in planning, but prompt to decide and act. His keen glance and quick movements expressed the energy he possessed in an unusual degree. He was direct and frank. As an illustration of these characteristics, he organized and conducted the Spy System for Grant and Sherman's Armies on the basis that spies should give correct information to the enemy, if any. A personality that made friends and a loyalty that bound them contributed in no small measure to his success.

Of modest and retiring disposition, he was dragged forward or pushed ahead by his friends, as his intimate history reveals, and this, of course, was the reaction of a loyalty that knew no bounds. It illustrates Emerson's maxim: "If you want a friend, be one." If a friend asked assistance, it was not "I will see what I can do," but "What can I do *now?*"

The history of General Dodge shows in a unique manner the value of preparedness. Educated in a military school at a time when war was not contemplated, and persisting in military training until the outbreak of the Civil War, he was fitted for military service at the start. His training in railroad and bridge building fitted him for the remarkable work of that character which he accomplished during the Civil War. His experience in freighting through an Indian country across the plains fitted him for the Indian campaign at the close of the war. His surveys west of the Missouri River and his wide knowledge of conditions in the West enabled him to advise President Lincoln on the selection from several routes proposed for the first transcontinental line.

All these experiences peculiarly fitted General Dodge to build the Union Pacific Railroad through a hostile Indian country with the necessity of transporting all materials for construction and supplies over long distances by boats and teams.

He died on January 3, 1916, in his eighty-fourth year, at his home in Council Bluffs, where he is buried.

General Dodge was elected Honorary Member of the American Society of Civil Engineers on March 2, 1915.

KENNETH ALLEN, M. Am. Soc. C. E.[1]

DIED SEPTEMBER 7, 1930

Kenneth Allen was born at New Bedford, Mass., on April 6, 1857, the oldest child of Edward A. H. and Eugenia (Teulon) Allen.

His father, Edward A. H. Allen, was graduated from the Rensselaer Polytechnic Institute, Troy, N. Y., in 1850, and from 1851 to 1855 was Professor there, in geology, physics, and chemistry. His ancestor, James Allen, came from England in 1639 and settled in a part of Dedham, Mass., which afterward was incorporated separately as the Town of Medfield. The Reverend

---

[1] Memoir prepared by George W. Fuller, William H. Breithaupt, and Arthur W. Tidd, Members, Am. Soc. C. E.

Jonas Clarke, of Lexington, Mass., prominent during the Revolutionary War, and whose house is now a museum, was his great-grandfather. Mr. Allen's mother, Eugenia Teulon Allen, was a daughter of Dr. William F. Teulon, Oxford graduate and descendant of a refugee Huguenot family which came to England in the first years of the Eighteenth Century. Dr. Teulon was married in London and, later, settled in Massachusetts.

After preliminary education at New Bedford, at Madison, Wis., where he spent a year (1872-73), at the preparatory school of the University of Wisconsin, and after a year at the West Newton, Mass., English and Classical School, Kenneth Allen entered Rensselaer Polytechnic Institute in the fall of 1874. He was graduated in the Class of 1879 with the degree of Civil Engineer.

Previous to his graduation Mr. Allen had been with the Boston Water-Works for more than a year. From 1879, he was engaged on railway work for three or more years, first on Western railways and then with the Pittsburg, McKeesport, and Youghiogheny Railway Company. In 1883, he was employed on surveys for the Water Department of Philadelphia, Pa., after which he again went West, on railway work, on branches of the Kansas City, St. Joseph, and Council Bluffs Railway Company and of the Missouri Pacific Railway Company, with both of which he had worked before.

From 1886 to 1890, Mr. Allen had an engineering office in Kansas City, Mo., in partnership with William H. Breithaupt, M. Am. Soc. C. E., with whom he was engaged on railway, general structural and bridge work, etc. On his return East in 1891, he was employed for a number of years with the Frick Coke Company, of Scottdale, Pa., largely on surveys.

He later served as Assistant Engineer to the Commissioner of Public Works, Yonkers, N. Y.; Assistant Engineer on the sewerage system of Williamsbridge, N. Y.; and, in 1895, he was in charge of studies for the sewerage system and sewage disposal of Baltimore, Md. From this time, his work was mainly on sewerage, in which branch of engineering he became prominent, and an authority. His Baltimore work was interrupted from 1900 to 1902 when he was in consulting practice with N. S. Hill and A. M. Quick, Members, Am. Soc. C. E., under the firm name of Hill, Quick, and Allen, of Baltimore and New York, N. Y., reporting on sundry water supply and sewerage projects. There was an interval also from 1902 to 1906 when he was Engineer and Superintendent of the Water Department of Atlantic City, N. J., but he returned to Baltimore in 1906 and until 1908 served as Engineer in Charge of the Low-Level Division of the Sewerage System.

In 1908, Mr. Allen was appointed Engineer of the Metropolitan Sewerage Commission of New York and, until 1913, was engaged in studies of New York Harbor phenomena and investigations of sewers and sewage disposal in the Metropolitan District. At various times during this period he was granted leave of absence for special consulting work, such as reporting on the sewage disposal of Hyndman and Bellwood, Pa., and on the East Orange-Newark (N. J.), storm-water, trunk sewer. During 1913, he inspected sewerage systems and sewage disposal works abroad, visiting England, Scotland, Holland, France, and Germany; some of his observations are incorporated in his book on "Sewage Sludge". On his return he resumed his investigations with the Metropolitan Sewerage Commission culminating in the development of a plan for the main drainage and sewage disposal of New York City. The work of the Commission was terminated in 1914, subsequent to which date the studies were continued by Mr. Allen as Sanitary Engineer on the staff of the Chief Engineer of the Board of Estimate and Apportionment, which position he held until, shortly before his illness, he was drafted for similar work in the newly-created Department of Sanitation of the City. Granted leave of absence during the World War, he served as District Engineer, United States Housing Corporation, in charge of seven housing projects in the Middle West.

Mr. Allen was employed as expert in the case of the "State of New York *vs.* State of New Jersey and Passaic Valley Sewerage Commission", involving the alleged pollution of New York Harbor by New Jersey sewage. He also gave expert testimony in several cases involving oyster pollution and garbage disposal and, from time to time, investigated and reported on numerous drainage, sewerage, and garbage problems in and about New York City.

Mr. Allen was the author of many papers and discussions in the scientific societies to which he belonged, and was a frequent contributor to the technical press. He had unusually wide contacts in the sanitary field both at home and abroad.

An operation in February, 1930, left him in impaired health from which he did not recover. A pathetic feature is that he had decided to retire from active work on October 1, 1930, and his friends had arranged to give him a formal testimonial of appreciation. He had, however, the satisfaction of seeing, shortly before he passed away, appropriations made for contracts for the construction of the 180 000 000-gal. daily activated sludge plant on Wards Island, the installation of which he had for years advocated and urged as constituting the key plant in the disposal of the sewage of New York City.

Mr. Allen was a member of Sigma Xi, the only one of his class at Rensselaer to receive such membership. He was a member of the American Society for Municipal Improvements, of the American Public Health Association, and of the Municipal Engineers of the City of New York. He was a member and first President of the New York State Sewage Works Association which he was largely instrumental in organizing. He served for several years as Chairman of the Society's Special Committee on Rainfall and Run-Off. He was also a member of the Unitarian Laymen's League, a Charter and Honorary Member of the Tramp and Trail Club of New York, in which he was very active, and a member of the Westchester Trails Association, and of the University Club of White Plains, N. Y.

Kenneth Allen, in 1886, was married to Rose Whitmore Switzer, of Philadelphia, Pa., who, with their two sons, Edward S. and Harold A., and their daughter, Mrs. Frances Allen Dorsey, survives him.

Of kindly and amiable disposition, which made him many friends, ever serene and unruffled, he was nevertheless a man of strong character, persistent in the accomplishment of a set purpose. Always interested in others, he was ever ready to give freely of his time and experience to help them in their troubles or problems. He retained his youthful enthusiasm and vigor to a quite unusual degree, and up to the time of his operation was a steady and welcome member on the regular trips of the hiking clubs to which he belonged.

Mr. Allen was elected a Member of the American Society of Civil Engineers on May 2, 1888.

WILLIAM FRANKLIN ALLISON, M. Am. Soc. C. E.[1]

DIED JULY 7, 1927

William Franklin Allison was born in Elgin, Minn., on March 31, 1870, the eldest child of William and Kezia (Culbertson) Allison. His parents moved with their family to South Dakota in 1877, where they encountered the usual hardships and difficulties of pioneer life in the Middle West. Here, Mr. Allison attended the rural schools during his early life, and, later, the grade and high schools in the villages adjoining the family farm.

He taught a number of terms in the district schools in order to provide the means for his attendance at the State College of Agriculture and Mechanic Arts, at Brookings, S. Dak. from which institution he was graduated with honors in 1895. In 1897, he received the degree of Bachelor of Science in Civil Engineering from Purdue University.

From Purdue Mr. Allison went back to his home where he was engaged in the practice of general surveying until the outbreak of the war with Spain, at which time he organized a company of infantry and was commissioned its Captain in the First South Dakota Volunteer Infantry Regiment. Soon afterward he was promoted to the rank of Major. His regiment was sent to the Philippine Islands early in the summer of 1898 and remained there for the duration of the war and during the Philippine Insurrection. Major Allison was the Engineer Officer of the Regiment (May, 1898, to October 1899), detailed for road and bridge building and for the laying out of earthworks—winning signal merit for his services. He was recommended for the Brevet Rank of Lieutenant-Colonel of Volunteers for distinguished gallantry.

After his return from the Philippines in October, 1899, Major Allison spent some time in general engineering in South Dakota and worked for a time for the Chicago and Northwestern Railway Company. Later, he took graduate work at the South Dakota State College and at Purdue University and spent a year in Cornell University. From Cornell he received the degree of Civil Engineer in 1904.

In September, 1904, he became an Instructor in Civil Engineering in the Colorado School of Mines, at Golden, Colo., and, in 1905, was advanced to the rank of Professor of Civil Engineering, and Director of the Department of Civil Engineering at this school, which position he filled for the ensuing seven years. During this period he was also engaged in considerable private practice, including municipal, highway, water-works, sewers, and irrigation projects involving an expenditure of approximately $1 500 000, besides the design and supervision of construction of a large testing plant for the School of Mines. He also served as City Engineer for Golden, from 1908 to 1911.

---

[1] Memoir prepared by C. Edward Magnusson, M. Am. Soc. C. E.

In 1912, Professor Allison resigned his position at the School of Mines and entered private practice as Consulting Civil and Sanitary Engineer, in Portland, Ore. where he remained until September, 1913, when he was appointed Professor of Hydraulics and Sanitary Engineering at the University of Oregon. In 1914, he left Oregon to accept the position of Professor of Municipal and Highway Engineering at the University of Washington, Seattle, Wash., which position he held until his death on July 7, 1927. From August, 1914 to May, 1917, he also served as Consulting Engineer for the State Board of Health of Washington.

From his early boyhood he was interested in military affairs. He enlisted in the National Guard of Dakota Territory at the age of sixteen, and served continuously until the outbreak of the Spanish-American War when he entered service with the Volunteers. At the beginning of the World War in 1914, he was intensely interested and felt that all who had been trained in military affairs should be ready to serve if the need presented itself. It was with this idea in mind that he offered his services to the Government and received the rank of Major in the Reserve Officers Engineering Corps of the United States Army. On the entrance of the United States into the war, he was called into service (May, 1917), and was granted a leave of absence from the University of Washington.

As Major he went to France with the American Expeditionary Forces in September, 1917, and from November, 1917, to April, 1918, he was assigned to duty with the Sixteenth Engineers on general construction work. From April to November, 1918, he was Chief of the Construction Staff at Is-sur-Tille, France, and during November and part of December, he was Consulting Engineer on water supply at Liffol-le-Grande. During the remainder of December and through January, 1919, he was Engineer with the Peace Commission in Paris, France. He was discharged from service in Washington, D. C., February 17, 1919, and returned to his duties at the University of Washington in March of that year.

Professor Allison took an active part in his profession as well as in church, business, and civic affairs. He served as President of the Western Washington Section of the American Society of Civil Engineers in 1923, and was a member of the Special Committee of the Society on Irrigation Hydraulics from 1922 to 1927. He was the author of a number of papers and bulletins on various phases of Irrigation and Sanitary Engineering. He was a member of the American Public Health Association; the Society for the Promotion of Engineering Education; the University Faculty Men's Club; the American Association of University Professors; the American Legion; and the Presbyterian Church.

On August 25, 1904, he was married to Mabel Hegeman, the daughter of Mr. and Mrs. P. J. Hegeman, of Brookings, S. Dak. He is survived by his widow; three children, Katherine, William, and Muriel; his mother, Mrs. Emma Allison, all of Seattle, and two brothers, Dr. H. T. Allison, of Stanwood, Wash., and A. C. Allison, of Lake Stevens, Wash., and a sister, Mrs. C. E. Winegar, of Sequim, Wash.

He was a peace-loving, genial, whole-souled man, loved and respected by all who were privileged to know him.

Professor Allison was elected an Associate Member of the American Society of Civil Engineers on October 4, 1910, and a Member on October 1, 1913.

JUSTIN KENDERDINE ANDERSON, M. Am. Soc. C. E.[1]

Died February 15, 1930

Justin Kenderdine Anderson was born in Sheffield, Pa., on September 26, 1869, the son of David Fort and Elizabeth (Kenderdine) Anderson. After graduating from the public schools, he received the degree of Bachelor of Science in Civil Engineering from Swarthmore College in 1889. He was a member of Delta Upsilon Fraternity.

Following his graduation, from 1889 to 1892, Mr. Anderson was engaged in construction work on the Clinch Valley Division of the Norfolk and Western Railroad, and, later, on the survey of lands, in Wayne, Mingo, and Logan Counties, West Virginia, for various coal companies. As a member of the firm of Forman and Anderson, his practice took him permanently to West Virginia where he held his first public office, from 1895 to 1896, as a member of the first Board of County Commissioners of Mingo County. During the next year he was Commissioner of School Lands in this County.

From 1899 to 1902, Mr. Anderson was Superintendent and Chief Engineer of the Guyandot Land Association holdings, covering more than 200 000 acres of coal, timber, oil and gas lands in Mingo, Wayne, Logan, and Lincoln Counties in West Virginia. He served as Assistant to the Chief Engineer of the Pocahontas Coal and Coke Company from 1902 to 1904, and during the following six years he was Superintendent and General Manager, as well as Chief Engineer and Director of several coal companies in the Pocahontas field. From 1911 to 1913, he was engaged in general engineering practice, with an office in Williamson, W. Va., and in the latter year was made the first Chief Engineer for the Public Service Commission of West Virginia, in which capacity he organized the Engineering and Inspection Department of the Commission. He retired from active service with the Public Service Commission, in 1919, but was frequently engaged by this body in a consulting capacity.

In 1913, Mr. Anderson established his home in Charleston, W. Va., where, in 1919, he opened an office as Consulting Engineer, specializing in public utility valuations and rate cases before the Public Service Commission. In 1920, he became senior member of the firm of Anderson and Taylor, Consulting Engineers. He frequently testified before the United States Supreme Court in interstate water power cases, and at the time of his fatal illness from heart disease, was at work on an investigation, for the Public Service Commission, of plans filed by the West Penn Power and Transmission Company in December, 1929, for an extensive water-power development program along Cheat River. In making that investigation he was aided by previous experience gained as one of a Commission of five members appointed by former Governor Morgan to study proposed water power legislation. He also investigated for the Commission, plans for water power projects in New River.

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

In recognition of his interest in military training Mr. Anderson was appointed a Civilian Aide to the Secretary of War in 1923, re-appointed in 1927, and was designated by former Governor Gore as a Colonel on the Governor's Staff. He was intensively interested in boys' activities and was a prominent figure at the first public investiture of the West Virginia Order of De Molay held at Clarksburg, W. Va. In addition, he was Chairman of the Advisory Board of the Charleston Chapter, Order of De Molay.

Mr. Anderson was a member of the American Institute of Mining and Metallurgical Engineers, President of the West Virginia Engineers Association, a member of the American Association of Engineers (Charleston Chapter), also a member of the Natural Gas Association of America, the West Virginia Oil and Natural Gas Association, and the Charleston Chamber of Commerce.

He was a Scottish Rite and a Thirty-second Degree Mason, and a member of the Beni-Kedem Shrine Temple. He was a charter member of Thacker Lodge 119, A. F. and A. M., and a member of the Clinch Valley Commandery, Knights Templars, at Tazewell Courthouse, Va. He held membership in the Edgewood Country Club, the National Advisory Board of Old Colony Club, the National League of Masonic Clubs, and the Rotary Club. He attended the Presbyterian Church.

He was married to Fannie Taliaferro Tinsley on February 26, 1895, and is survived by his widow and two children, Seaton Tinsley and Frances Gaines Anderson. The oldest son, Lane Schofield, was killed while serving overseas during the World War and posthumously received the Distinguished Service Cross for bravery in action.

Mr. Anderson was elected a Member of the American Society of Civil Engineers on September 2, 1914.

HENRY CLARKE ASH, M. Am. Soc. C. E.[1]

DIED JUNE 20, 1929

Henry Clarke Ash, the son of John and Sarah (Clarke) Ash, was born on October 27, 1864, at Preemption, Ill. He was of Scotch-Irish ancestry.

He attended Cornell College, at Mount Vernon, Iowa, where he received the degrees of Bachelor of Civil Engineering, in 1890, and Civil Engineer in 1897.

On the completion of his studies in 1890, Mr. Ash went to work with the Chicago and Northwestern Railway Company in the capacity of Rodman and

---

[1] Memoir prepared by a Committee of the Duluth Section consisting of W. A. Clark, M. Am. Soc. C. E., and G. H. Butler, Sr., and Sheldon B. Shepard, Assoc. Members, Am. Soc. C. E.

Leveler, but left this Company in 1891 to become Assistant City Engineer, of Duluth, Minn. While serving in this position, he was in charge of the office and field work in the West End Branch of the City Engineering Department, from which he supervised the customary engineering work necessary to such an office. He carried on the construction of about nine miles of street paving, six miles of sewers, two pile and timber trestles, two miles of rubble masonry retaining walls, and a great deal of ordinary minor city construction, such as sidewalks and curb and gutter installation.

In 1896, Mr. Ash joined the engineering forces of the Duluth and Iron Range Railroad Company, and was in charge of the field party at Two Harbors, Minn. He laid out the new yard system for handling and classifying the ore trains for the docks and located the "Loop Line" from the yards to the main line. He also re-surveyed and placed monuments for North Two Harbors and, later, made a complete survey of all the property owned by the Railroad Company in Duluth. In addition, he located the piers for the construction of the Duluth-Superior Interstate Bridge and inspected the foundation work for this structure.

In 1896 he returned to Cornell College and for a year was an Instructor in "Adjustment, Use, and Care of Instruments", and in "Field Practice". During this period Mr. Ash carried on the necessary post-graduate work to receive his Civil Engineer degree which was presented to him in June, 1897.

On examination he became eligible for appointment to the grade of United States Junior Engineer, and in June, 1897, received his assignment as United States Inspector in the Chicago District under Maj. W. L. Marshall, Corps of Engineers, U. S. A. From June, 1897, to the fall of 1898, Mr. Ash was engaged as Inspector of Dredging in the vicinity of Chicago, Ill. He was then transferred to Sterling, Ill., for work on the Illinois and Mississippi Canal. He carried on the office work and surveys for about one year and then became Resident Engineer on the Canal in charge of the Tampico Office and Receiving Station. During this time, approximately eleven miles of the Feeder Canal were completed, including foundations for siphon culverts, iron and concrete culverts, bridges, and bridge approaches.

In 1902, Mr. Ash became associated with the Great Northern Power Company (now the Minnesota Power and Light Company of Duluth) for which he made an examination and report on a water power proposition in Michigan. Following this work he made the topographical surveys, current meter and weir measurements of the St. Louis and Cloquet Rivers, and the necessary calculations of flow in rivers, canals, and pipes with the fluctuations of head incidental to preparations for the design of the Thompson Dam and Reservoir at Thompson, Minn. Mr. Ash designed the dam for this Company and it was his privilege to supervise the construction work on the Thompson Reservoir consisting of: An earth dam, ½ mile in length, faced with rip-rap; a 700-ft. rock-fill dam with a concrete core wall; a concrete arch dam, 55 ft. high, with a 100-ft. radius; a concrete over-fall dam, 370 ft. long, which was 45 ft. high; 1 000 lin. ft. of concrete retaining dams ranging from 10 to 50 ft. in height; reinforced concrete sluice-gate house and head-gate house; 1 mile of canal;

concrete and steel wagon bridges; an 80-ft. steel girder railroad bridge; and supervision of a complete laboratory for testing materials for the entire development.

In 1928, Mr. Ash prepared a paper,[2] for the Twenty-fifth Convention of the American Concrete Institute held in Detroit, Mich., February 12 to 14, 1929, covering this work. Mr. M. B. Lagaard, of the Portland Cement Association, made an examination of this concrete work during 1928, and presented a discussion[3] on the paper by Mr. Ash, pointing out the fact that the thoroughness of the methods used by him as Resident Engineer was accountable for the excellent condition of all the concrete structures.

From 1907 to 1910, Mr. Ash was with the Minnesota and Ontario Power Company, following which he was Consultant on the Rice Lake Dam. In the winter of 1910 and 1911, he was in Beaudette, Minn., in charge of relief work following the disastrous forest fires. From 1915 to 1921, he was engaged as Assistant Engineer with the Duluth, Missabe and Northern Railway Company on valuation work. He then became Executive Secretary for the City Planning Commission of Duluth.

Mr. Ash had an unfailing sense of humor, which made him a delightful companion and associate. He was in very poor health during the last two years of his life, and while suffering and sick, as he knew himself to be, his usual cheerful smile gave an outward sign of content and happiness. His life was a continuous portrayal of that which man should be toward his fellows—considerate of others even to the neglect of self. It may be compared with that in which it was most useful, a piece of construction in which all success depends upon the foundation. The life of Mr. Ash on this earth constitutes the foundation upon which the superstructure of the most marvelous future may be erected without fear of deterioration due to faulty substructure.

Mr. Ash was a member of the Glen Avon Masonic Lodge, and of the Scottish Rite Fraternity. He was also a member of the Glenn Avon Presbyterian Church, in which he took an active interest.

On January 11, 1899, he was married to Frances C. Hoadley, of Boston, Mass., who, with a son, James H., and a daughter, Katherine C., survives him.

Mr. Ash was elected a Member of the American Society of Civil Engineers on October 3, 1906.

FRANK MILLIGAN ASHMEAD, M. Am. Soc. C. E.[4]

Died December 7, 1925

Frank Milligan Ashmead was born at Germantown, Pa., on February 23, 1853, the son of Samuel B. and Mary (Milligan) Ashmead. His father, Samuel B. Ashmead, a leather merchant in the City of Philadelphia, Pa., traced his

---

[2] "The Thompson Dam and Reservoir" (Minnesota), *Proceedings*, Am. Concrete Inst., Vol. XXV, 1929, p. 29.

[3] *Loc. cit.*, p. 39.

[4] Memoir prepared by James H. Herron, M. Am. Soc. C. E.

American ancestry back to John Ashmead who came to Germantown, from Cheltenham, England, in 1682.

Mr. Ashmead received his early education in the public schools of Germantown and Philadelphia, afterward attending the Philadelphia Polytechnic Institute from which he was graduated in 1875.

His first professional engagements were in construction work on the Dutchess and Columbia Railroad, in New York State, and the Columbia and Port Deposit Railroad in Pennsylvania. He afterward went to the Allegheny Valley Railroad Company with which road, its branches, and successors, he remained during his professional life. He first served as Assistant Engineer on the low-grade division, after which he was, successively, Engineer, Department of Bridges, Resident Engineer, and Chief Engineer, with headquarters in Pittsburgh, Pa.

In 1901, when the Pennsylvania Railroad Company took over the Allegheny Valley Railroad and merged it with the Western New York and Pennsylvania Railroad, designating them the Buffalo and Allegheny Valley Division, Pennsylvania Railroad, Mr. Ashmead became Engineer, Right of Way, and Assistant Engineer of the Division. He remained in Pittsburgh until 1903, when he was transferred to Buffalo, N. Y., as Assistant to the Principal Assistant Engineer.

In 1918, Mr. Ashmead's health began to fail, and by request he was transferred to Pittsburgh (to be near his children), where he continued to serve as Assistant to the Principal Assistant Engineer intermittently on problems of right of way, etc., until he retired in 1921. After this time he lived, successively, in Atlantic City, N. J., and Cleveland, Ohio, in which latter city he died on December 7, 1925.

Frank Ashmead was a man of sterling integrity, retiring in disposition and much respected and beloved by his subordinates and associates. He was well informed, not only in regard to his profession, but upon many other subjects of a philosophical and economic character. While he spent practically all his professional life with one organization, he was well and favorably known on many of the Eastern railroads.

He was married on December 22, 1881, to Mary Lillian Herron, of Oil City, Pa., who with a son, Charles B. Ashmead, and a daughter, Edith (Ashmead) Hutchinson, survives him.

Mr. Ashmead was elected a Member of the American Society of Civil Engineers on July 6, 1887.

FRANK FOY AXTELL, M. Am. Soc. C. E.[1]

DIED AUGUST 14, 1929

Frank Foy Axtell, the son of Thomas Jefferson and Edna Ann Axtell, was born at Washington, Ind., on August 18, 1867. His father was a successful business man, having come to Indiana from his native State, Pennsylvania, as a young man. The mother was from one of the Indiana pioneer families long resident in that part of the State and their son grew up with many early advantages in a material and social way.

---

[1] Memoir prepared by Charles Warren Hughes, Assoc. M. Am. Soc. C. E.

Following his graduation from the local High School, Mr. Axtell entered Indiana State University, from which institution he received his degree in 1891. In September following, he was temporarily employed as Rodman on a precise level survey of the Missouri River. He left that work in the spring of 1892 to go as Recorder on a topographical survey of the Mississippi River, under the Mississippi River Commission, at St. Louis, Mo., becoming Field Draftsman in 1894. In that same year he left the work of the Commission for a time, engaging in the survey of suburban property and on an interurban railway in St. Louis, with F. B. Maltby, M. Am. Soc. C. E.; in 1895 he went with Mr. Maltby to Uvalde and Eagle Pass, Tex., on an irrigation project. On returning to the Mississippi River Commission in 1896, he was, successively, Recorder, Draftsman, Leveler, and Surveyor. He was promoted to the grade of United States Surveyor in 1897, and given charge of a Topographical Party.

As early as 1899, however, it became so advisable for Mr. Axtell to seek a milder climate on account of his health, that January, 1900, found him in New Orleans, La., attached to the United States Engineer Department, under the late Lt.-Col. H. M. Adams, Corps of Engineers. He was sent to Calcasieu Pass, La., to supervise the reconstruction and extension of the jetties at that place, covering mattress foundation and rip-rap stone work, which involved considerable expense. On the completion of this work, he was for a time Assistant to the late Mr. C. Donovan, at Port Eads, La., at the mouth of the Mississippi River. In the fall of that year Mr. Axtell transferred to Pilot Town, La., where he supervised the construction of a submerged dam across Pass à Loutre, one of the mouths of the Mississippi River, a work of many difficulties requiring the expenditure of $175 000.

On the completion of this work, Mr. Axtell was transferred back to the U. S. Engineer Office, at New Orleans, and advanced to the grade of U. S. Junior Engineer. He was placed in charge of the improvement of a number of small streams, including the dragging and dredging of these and the planning of improvements of a permanent nature. Then, for a time, pending the passing of the River and Harbor Bill, he was employed in the Division Engineer's Office of the Louisville and Nashville Railroad Company, at Montgomery, Ala. Following the passage of the River and Harbor Bill in June, 1902, Mr. Axtell returned to New Orleans and in December, 1902, he was transferred to Sabine, Tex., where he was given local supervision of the improvement of Sabine Pass Harbor and Jetties, Sabine Lake, and the Sabine and Neches Rivers. In 1906, the U. S. Government acquired title to the Port Arthur Ship Canal and Harbor, and the supervision of those works was turned over to Mr. Axtell. In 1908, the office maintained by him at Sabine was moved to Port Arthur, Tex.

From 1902 to 1912 his duties included the following: (1) Complete triangulation, topographic and hydrographic surveys of Sabine Pass, Sabine

Lake, and Sabine and Neches Rivers; (2) the preparation of plans and specifications and the supervision of jetty work at Sabine Pass, in 1903, 1905, 1908, and 1910, to the amount of $716 000, the work including rip-rap stone work foundation, and the granite and concrete capping; (3) the surveys for, the preparation of plans and specifications, and the supervision of the work of excavating, a canal connecting the Port Arthur Ship Canal and the mouths of the Sabine and Neches Rivers, known as the Sabine-Neches Canal (this canal had a depth of 10 ft., was 100 ft. wide on the bottom, and 16 miles long when first completed, but it has been widened and deepened several times since as appropriations became available); (4) the supervision of the dredging and the repairing of the U. S. hopper, sea-going dredge, *Sabine*, and the U. S. hydraulic dredge, *Port Arthur;* and (5) the supervision of the surveys for the Intracoastal Canal from (*a*) Calcasieu River, Louisiana, to Sabine River, Texas; and (*b*) Sabine Pass, Louisiana and Texas, to Galveston Bay.

In 1912, Mr. Axtell went with the Texas Company at Port Arthur, as Construction Engineer, in which position he supervised the construction of concrete retaining walls, docks, warehouses, and oil pipe lines. In 1913, he was sent to Tampico, Mexico, where as Chief Engineer for the Panuco Transportation Company he was engaged in land surveys, hydraulic dredging, and the construction of docks, warehouses, and dwellings. When the Panuco Transportation Company was merged into the Texas Company of Mexico, he was made Chief Engineer for the construction of oil pipe lines, boiler and pump houses, docks and dwellings, hydraulic dredging, and filling—a total of more than $1 500 000. He remained in Mexico seven years and returned in 1920, as at this time his two sons were of an age to need the educational opportunities of the United States.

In 1921 Mr. Axtell became allied with the Gulf Refining Company, of Port Arthur, reputedly the largest in the world. Dredging, the filling in of a bayou, a large expansion and construction program carried on by the Gulf Company, were projects in which he had a part; later, the land maps and records were handled. In spite of his failing health, for more than a year, he kept actively engaged with whatever came in his line of duty until July 19, 1929, when it became impossible for him to continue. He went home a sick man, indeed, and was unable to leave his bed from that time. Four weeks from that day, on August 16, he was laid to rest in Greenlawn Cemetery, Port Arthur, the city which had been his home for more than 20 years, except the 7-year period he spent in Mexico.

Mr. Axtell was a man of rare personality, loyal to his friends, yet in his judgment charitable toward all. He was a devoted family man—a man of high ideals. He was married on September 15, 1904, to Harriet Duncan, of Washington, Ind., who, with their two sons, Edwin Duncan and Frank F., survives him. He is also survived by one sister, Mrs. George Kramer.

He became a member of the Presbyterian Church in early manhood and continued in that faith to the end of his life. He took great interest in civic affairs as long as his health permitted, having served as a member of the local Chamber of Commerce and other organizations.

His technical training made him accurate in his methods, and he was unsparing of himself as an Engineer. He was generous, to a fault, toward those with whom he was associated in various engineering projects, and this to such an extent that often others reaped where he had sown.

Mr. Axtell was elected an Associate Member of the American Society of Civil Engineers on June 6, 1911, and a Member on November 26, 1918.

MORTIMER GRANT BARNES, M. Am. Soc. C. E.[1]

DIED OCTOBER 7, 1930

Mortimer Grant Barnes was born on a farm near Reedsburg, Wis., on January 17, 1867. His parents were James B. and Alice Randall Barnes, who, when the boy was very young, moved to Nebraska, where he grew up and received his early education. In 1888 and 1889 he attended a business college at Holton, Kans., where he also took a course in higher mathematics. He then entered engineering work, and, in 1891, was elected County Surveyor of Boone County, Nebraska.

In 1892, Mr. Barnes entered the Engineering Department at the University of Michigan. During the next four years, he spent about half his time in the employ of the Corps of Engineers, U. S. Army, on the construction of the Poe Lock of the Sault Ste. Marie Canal, Michigan, and the other half in study at the University, from which he was graduated in 1896 with the degree of Bachelor of Science. After his graduation, he pursued the advanced course in mathematics, majoring in hydraulics and masonry; he received the degree of Civil Engineer in 1901. In 1922, the University of Michigan conferred upon Mr. Barnes the degree of Master of Engineering.

In 1896 and 1897 he was Assistant Chief Engineer of the Lake Superior Power Company, at Sault Ste. Marie, Mich., on the construction of a power plant at that place. For the next three years he was making surveys and preliminary plans for a canal in Alabama and was engaged on the survey and study of the United States Deep Waterway Survey in New York State and Canada. From 1900 to 1905, he was in charge of the design and construction of the Hennepin Canal in Western Illinois. In 1905, he was transferred to Rock Island, Ill., to take charge of the design and construction of the lock on the Mississippi River at Moline, Ill.

In 1906, Mr. Barnes was placed in charge of the design of locks and other structures of the Panama Canal, but resigned in 1907 when the U. S. Army Engineers were given charge of the work there. He then entered the employ of the Board of Water Supply, New York City, studying the design and foundation in connection with the construction of the Catskill Water Supply, but in a few weeks he was appointed by Governor Hughes as Consulting Engineer on the New York State Barge Canal. He remained on that work for about eight years. During a part of that time, he also had a private office in Albany, N. Y., where he carried on a general engineering practice, having been engaged principally in problems of navigation, terminals, and the design and construction of water-power plants.

---

[1] Memoir prepared by Walter M. Smith, M. Am. Soc. C. E.

During the major part of 1914 he was associated with Walter M. Smith, M. Am. Soc. C. E., in the general practice of hydraulic engineering, under the firm name of Barnes and Smith, with offices in Albany. During this time, preliminary designs were made for power plants at various places and plans were completed for locks for the Brazos and Trinity Rivers, in Texas, and for a hydro-electric plant at Waterloo, N. Y. This partnership was dissolved in the fall of 1914 on account of the exigencies of the World War. Mr. Barnes continued in general practice, constructing the plant at Waterloo, and designing and building another at Seneca Falls, N. Y.

In 1917, Mr. Barnes was called to Illinois by Governor Frank O. Lowden to take charge of the design and construction of the Illinois Waterway as Chief Engineer of the Illinois State Division of Waterways. A $20 000 000 bond issue had been voted for the undertaking several years before this time. The waterway was to extend from the west end of the Sanitary District Canal, at Lockport, to the head of navigation on the Illinois River, at Starved Rock, Ill. Several attempts had been made by previous State Administrations to design a waterway acceptable to the United States War Department by which it had to be approved, but all such plans had failed. Mr. Barnes, however, designed a waterway that was approved by the Department, and work was started in 1919. He continued to be Chief Engineer until May, 1928, when he had to relinquish the arduous duties of that office on account of declining health, but he was retained by the State in a consulting capacity. After a long rest, his health was sufficiently improved to allow him to do consulting work, and he was engaged by the States of New Jersey and Pennsylvania to assist in opposing New York City in diverting water from the Delaware River for its own use. His strenuous work in this suit again caused a steady decline in health, from which he was not able to rally.

In addition to the work herein outlined, Mr. Barnes was consulted by various municipalities and other bodies at various times. He also owned and had most of the managerial cares of a large cattle ranch in Nebraska.

He suffered greatly from ill health for many years before his death, due largely to his intense interest in his work and his unwillingness to take rest or allow himself to rest while engaged on important tasks.

In 1898, Mr. Barnes was married to Mina M. Wood. They had three children, Alice E., Florence L. (now Mrs. Oscar C. Strom), and James M., all of whom survive him.

Mr. Barnes was elected an Associate Member of the American Society of Civil Engineers on November 2, 1898, and a Member on May 31, 1904. He was also a member of the Western Society of Engineers.

AXEL SAMUEL FREDERICK BERQUIST, M. Am. Soc. C. E.[1]

Died October 6, 1915

Axel Samuel Frederick Berquist was born on February 1, 1863, in Smaland, Sweden. In 1882, he entered the University of Lund, Sweden, from which he was graduated in 1886.

Subsequent to his graduation, Mr. Berquist came to the United States and was engaged with the Baldwin Locomotive Works until 1892 in the capacities of Machinist, Draftsman in the Highway Department, Tool Designer, and Erection Superintendent.

From 1892 until 1897, he acted as Chief Draftsman and Engineer in the Construction Department of the Spreckels Sugar Refining Company, in Philadelphia, Pa. Among the principal structures which Mr. Berquist erected are the following: A new raw sugar wash house, complete with machinery; Pier 61 on the Delaware River, with sheds for this pier and also for Pier 60; a sugar warehouse, 500 by 175 ft., with 2 miles of over-head track for trolley operations; a new style of furnace for burning sugar baskets and mats; ten bridges, from 50 to 98 ft. in span, for various purposes; and, finally, he assisted in remodeling the entire plant of the Spreckels Company.

In 1897, Mr. Berquist entered the service of the American Coffee Company with which he remained as Engineer until 1900. He was instrumental in designing and erecting the Company's plant in Brooklyn, N. Y., and in connection with a power house for this plant, he invented the "Berquist Suspension Bunker" for storing coal, ore, and similar materials.[2]

From 1900 until a short time prior to his death, Mr. Berquist was associated with the Rapid Transit Subway Construction Company of New York, N. Y. As Engineer he was in charge of metal structures and the steel erection, including the new Harlem Ship Canal Bridge, at Kingsbridge, N. Y. His work likewise included all revisions and alterations of steel work at the time of, and during, erection and the financial adjustments relating thereto. It was a conspicuous fact that in his dealings representing the Contractor with the several sub-contractors, his decisions were uniformly fair and diplomatic. After severing his connection with subway work, Mr. Berquist was engaged in consulting practice and in some contracting. He was associated with the work of track elevation of the New York Central Railroad through the City of Yonkers, N. Y., and, at this time, he also continued to be actively engaged in the development and improvement of his "suspension bunker."

His untimely death removed from the ranks of the Engineering Profession an able and an experienced engineer. He was of a quiet but sociable disposition, agreeable, patient, and ever ready to help solve the mathematical and engineering problems which his co-workers might bring to him. Early in the introduction of reinforced concrete, he devised some very workable and simple formulas for practical application.

---

[1] Memoir compiled from information furnished by the late William J. Boucher, M. Am. Soc. C. E., and Theodore Belzner, Affiliate, Am. Soc. C. E., and from data on file at the Headquarters of the Society.

[2] *Engineering News*, July 27, 1899, p. 54, and *Engineering News-Record*, August 31, 1922, p. 346.

Mr. Berquist was elected a Member of the American Society of Civil Engineers on June 6, 1906.

ALEXANDER LESLIE BLACK, M. Am. Soc. C. E.[1]

Died March 3, 1930

Alexander Leslie Black was born in New Orleans, La., on January 4, 1871. He attended private preparatory schools in New Orleans and in New York, N. Y., and was graduated from the School of Mines, Columbia College, in 1890. He then engaged in mining work in the West and in Mexico for three years, subsequently acting as Assistant Engineer on the topographical survey of New Orleans.

In 1894 and 1895, Mr. Black had charge of the reconstruction for electric operation of the St. Charles Street Railroad in New Orleans, and remained with that Company supervising operation and construction extensions for this and other local companies until 1901, when the New Orleans properties were consolidated. He then became Engineer for the New Orleans Railway and Light Company and its predecessors controlling the New Orleans street railways and lighting companies.

In 1910, he became identified with the firm of Ford, Bacon, and Davis, Incorporated, as Engineer in charge of Southern Properties, supervising the reconstruction, maintenance, and operation of street railway, electric light and power, and water and gas properties in New Orleans, Birmingham, Ala., Memphis, and Knoxville, Tenn., Little Rock, and Pine Bluff, Ark., and Houston, Tex. These activities engaged Mr. Black's time until 1914, when he became Chief Engineer of Design and Construction of public cotton warehouses and terminals in New Orleans. This work was highly diversified in character, including the construction not only of warehouses, but of power plants, cotton compresses, wharves, tracks, cotton-handling equipment, and land reclamation.

Two years later, Mr. Black was made Chief Engineer on Design and Construction of the New Orleans Public Grain Elevator. This project included a warehouse, grain dryer, million bushel storage bins, with wharves, conveyors, galleries, trackage, and land reclamation. During the eight years following this engagement (which in itself consumed only one year), Mr. Black was occupied with valuation and rate negotiations for the Market Street Railway Company, Sierra and San Francisco Power Company, Coast Valleys Gas and Electric Company, in San Francisco, Calif., and as Consulting Engineer, Way and Structures, for the Market Street Railway Company. From 1924 to 1925, he was Vice-President in Charge of Market Street Railways in San Francisco. The two years following he was Chief Engineer of the New Orleans-Pontchartrain Bridge, conducting investigations for the foundations, design, and construction of a bridge, with highway approaches, over Lake Pontchartrain.

---

[1] Memoir prepared by William von Phul, M. Am. Soc. C. E.

Failing health caused Mr. Black to curtail his activities considerably during the three years before his death, and he alternated between the Corporation's New York and Birmingham Offices. His last assignment was as Consultant during the construction of the transportation system of the Southern Natural Gas Company.

He was a member of the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, the American Association of Engineers, and the Louisiana Engineering Society.

He is survived by his widow, the former Virginia Bullitt Grant, and two sons, William Grant and Alexander Leslie Black.

Mr. Black was elected a Member of the American Society of Civil Engineers on December 4, 1907.

JOHN CARLISLE BLAND, M. Am. Soc. C. E.[1]

DIED JULY 16, 1927

Champion of riveted against pin-connected bridges, successful constructor of many bold and difficult works, and a leading figure in the art during more than thirty years, John C. Bland, long the Bridge Engineer of the Pennsylvania Lines West, left a lasting impress on bridge engineering. His death in 1927 deprived railroad bridge construction of one of its most original and creative minds.

John Carlisle Bland was born in Philadelphia, Pa., on January 22, 1853, of British West Indies parentage. After an education in Mathematics at Johns Hopkins University, he began practical work with the Pennsylvania Railroad Company in 1871, first as member of a survey party and later as Assistant Engineer in charge of a bridge under construction. After three years, however, he was engaged as Engineer by the bridge building firm of J. H. Cofrode and Company, Philadelphia. He remained with this firm for six years, and then spent three years (1879-1882) as Engineer in the office of the Supervising Architect of the United States Treasury. He then returned to the firm, which had become Cofrode and Saylor, Philadelphia Bridge Works, and remained with it until 1888. In the latter year ill health, which had for some time troubled him, compelled him to move permanently to Colorado, where until 1894 he practiced as Consulting Engineer in Colorado Springs, serving, during this period, the Colorado Midland Railway and the Denver and Rio Grande Western Railroad Companies.

Mr. Bland's second and permanent connection with the Pennsylvania Railroad Company began in 1895. A year earlier he had taken charge for the late Thomas Rodd, M. Am. Soc. C. E., of the construction of the East Pittsburgh Plant of the Westinghouse Electric and Manufacturing Company, but soon he was called by the late M. J. Becker, M. Am. Soc. C. E., Chief Engineer of the Southwest System, Pennsylvania Lines West, to design and supervise the construction of the Company's Ohio River Bridge, at Cincinnati, Ohio. Upon completion of this work he was placed in charge of all bridge work on the Southwest System. This was the beginning of an unbroken career of important bridge construction for the Pennsylvania System, which continued until his retirement for age in 1923. He held, successively, the positions of Principal Assistant Engineer of the Southwest System (1896 to 1900); Engineer of Bridges of both the Northwest and Southwest Systems (1901-1920); and Engineer of Bridges and Buildings. After his retirement he lived in Atlantic City, N. J., where he died.

---

[1] Memoir compiled from information on file at the Headquarters of the Society.

Mr. Bland's first great contribution to railroad bridge practice was the reform of designing and contracting methods prevalent in the Eighties and early Nineties. It was usual for the railroad companies to prepare a general specification and a plan of span lengths, and to require bidders to prepare designs and submit lump-sum bids. This practice, while it stimulated the ingenuity of bidders and resulted in low first cost, also produced many poor bridges. Mr. Bland strongly opposed this practice, and for the first time at Cincinnati prepared a complete general design as the basis for bids. This innovation led ultimately, through his own influence as well as through the precedent of good practice which it set, to the progressive abandonment of the old method by all railroad companies. The change cleared the way for the great advance in the art which growing railroad requirements demanded.

In the same period, Mr. Bland did much to extend the use of steel in place of wrought iron to all railroad bridge work. The Cincinnati Bridge was built wholly of steel, but the Northwest System of the Pennsylvania Lines West continued to restrict steel to eye-bars, rods, and pins. When Mr. Bland became Bridge Engineer of both Systems he prepared a new specification which permitted the use of steel throughout.

Much more important, however, was his influence in turning railroad bridge practice away from pin-connected construction toward full riveted-truss construction, now the established standard in the railroad bridge field. In the first large structures which Mr. Bland designed and built for the Pennsylvania Lines West, he adopted pin-connected trusses, the last important one being the Panhandle Bridge, over the Monongahela River at Pittsburgh, Pa. (1903), comprising spans from 183 to 363 ft. long. The 230-ft. riveted spans of the Ohio River Bridge, at Steubenville, Ohio, were his first important riveted trusses. Soon it was shown that riveted construction could so thoroughly meet the requirements of modern engineering, and was so well adapted to the improved facilities of fabrication and erection, that in a relatively short time it was generally adopted. Nevertheless, Mr. Bland's own works still remain among the largest and heaviest riveted spans, notably the 416-ft. and 525-ft. channel spans of the Ohio Connecting Railroad over the Ohio River at Pittsburgh (1913-14), and the 644-ft. channel span of the Louisville, Ky., Bridge (1916-18).

Because of their place in the record of heavy railroad bridge construction, a few of Mr. Bland's larger undertakings for the Pennsylvania Lines West claim mention here: The Allegheny River Bridge, at Pittsburgh (1902), a most difficult replacement undertaking, involving what is still the heaviest bridge of its size; it has a 333-ft. channel span, and four lines of trusses, carrying five tracks on two decks; not only was it built under traffic, but in 1917 it was raised, also under traffic, about 13 ft. to give greater under-clearance for river traffic; both the original construction and the raising were operations of utmost complexity and delicacy, and both were carried out with complete success, free from all untoward incident: The Monongahela River Bridge, at Pittsburgh (1903), channel span, 363 ft.: The Ohio River Bridge, at Steubenville (1909), the replacement of an old pin-connected bridge under traffic, with change of span lengths to give a wider channel; the channel bridge, a three-span, continuous-riveted structure of 353, 568, 353 ft., respectively, was not built until 1926-1927; the 230-ft. spans of this bridge (1909) were Mr. Bland's first large riveted trusses: The Ohio Connecting Railroad Bridge, at Pittsburgh (1913-14), through channel spans of 416 and 525 ft., and eleven deck spans of 171 ft., the longer channel span being erected without falsework, by cantilever construction using parts of the other channel span as counterbalance: The Muskingum River Bridge, at Tyndall, Ohio (1914), one of the early examples of rolling a series of spans into place laterally between trains without delay to traffic; the previous bridge had been washed out in the great 1913 Central States flood: The Calumet River and South Branch (vertical lift) Bridges, at Chicago, Ill. (1914-15); two double-track lift spans of 210 ft. over the Calumet, and one double-track lift span of 273 ft. over the South Branch, probably the heaviest railroad lift span attempted up to this time; the Miami River Bridge, at Dayton, Ohio (1916); three double-track spans of 221 ft., rolled laterally to place in two successive operations because of complicated approach-track conditions; the Ohio River Bridge at Louisville (1916-18); this structure comprises twenty-two spans, ranging from 110 ft. to 644 ft. long, and replaced a historic bridge built by the late Albert Fink, Past-President, Am. Soc. C. E., in 1868-70; Mr. Bland's 644-ft. channel span is the longest riveted simple truss for railway use yet constructed.

Outside of bridge work, Mr. Bland was consulted by the Pennsylvania Lines West on many problems of railroad construction and maintenance, chiefly because of his mathematical insight. He made many investigations and reports on such questions. He was also called upon to act as Consulting Engineer to the Terminal Railroad Association of St. Louis on the Eads and Merchants Bridges. In this service he made a thorough investigation of the old Eads Bridge, built in 1874, and strengthened it for modern traffic. He later similarly investigated and reinforced the near-by Merchants Bridge.

His published works include one of the early handbooks for engineers and architects, that of the Pottsville Iron and Steel Company, of Pottsville, Pa. (1887). He contributed discussions on "Train Loadings for Railroad Bridges"[2] and on "Some Disputed Points in Railway Bridge Designing,"[3] two papers published by the Society, as well as similar articles to other scientific societies.

---

[2] *Transactions*, Am. Soc. C. E. Vol. XXXI (1894), p. 194.

[3] *Loc. cit.*, Vol. XXVI (1892), p. 232.

His interests, however, were not limited to the field of bridges. He was a man of broad general culture and studied deeply many scientific subjects as well as cultivating philosophy, literature, and humanistic studies. In particular, he was a brilliant mathematician, and both pure and applied mathematical investigations were his principal hobby.

Mr. Bland was married to Emma Virginia Long, of Philadelphia, in 1874. He is survived by one son, Arthur Stanley Bland, of St. Louis, Mo.

Mr. Bland was elected a Junior of the American Society of Civil Engineers on May 12, 1875, and a Member on June 4, 1879.

GEORGE BOWERS, M. Am. Soc. C. E.[1]

Died February 28, 1930

George Bowers was born on October 30, 1848, in Middlesex Village, then a part of Chelmsford, Mass., the son of Sewall and Sylvia (Fisher) Bowers. He was of English ancestry; his earliest known paternal ancestor was Michael de Bures who was seated in Dorset, England, in 1075. His earliest paternal ancestor in America was George Bowers, whose son, Jerathmell Bowers, built the Bowers homestead in 1695 on land granted from the King of England. Mr. Bowers lived in this same house many years, the home still remaining in the Bowers family, having been passed down from generation to generation.

George Bowers spent his early life in Middlesex Village and Lowell, Mass., where he attended the local schools. He started his engineering work in the City Engineer's Office in Lowell, remaining for two years before entering the Massachusetts Institute of Technology with the Class of 1875. After leaving the Institute he was employed as First Assistant Engineer on the construction of the Waltham, Mass., Water-Works. Returning to Lowell, he became First Assistant Engineer and, later, City Engineer.

In addition to his regular work as City Engineer, Mr. Bowers carried on an extensive series of experiments with driven wells to determine whether it would be possible to supply Lowell with driven-well water. He was convinced that this could be done, and his judgment proved to be correct, for since the wells were put into operation, Lowell has used no other water supply. He was the author of a number of papers on his driven-well experiments.

After leaving the City Engineer's Office, Mr. Bowers took up consulting work, which he followed until his last illness. He planned, and was Engineer during the construction of, a new driven-well water supply for Chelmsford, Mass., and acted as Consulting Engineer for The Nashua Manufacturing Company, and The American Hide and Leather Company in connection with water supply problems and the disposal of wastes. During the World War, Mr. Bowers was Engineer for a development for the United States Housing Board.

At a special meeting of the Board of Trustees of the Lowell Institution for Savings, of which Mr. Bowers was First Vice-President, the following memorial was placed on the records:

---

[1] Memoir prepared by E. B. Carney, Treas., Lowell Inst. for Savings, Lowell, Mass.

"George Bowers, our First Vice-President and Chairman of our Board of Investment, died Friday morning, February 28, 1930, at a few minutes after midnight. He was born October 30, 1848, in Middlesex Village, and afterwards lived in the old Bowers Homestead on Wood Street, built in 1695. This is supposed to be the oldest house now in Lowell.

"He was elected Corporator, Trustee, Vice-President, and member of the Board of Investment in 1904. Mr. Bowers had been City Engineer for over twenty years, but in 1911 was defeated for re-election.

"No ruler ever had his subjects more at heart, nor has a great general looked after his soldiers one bit more than Mr. Bowers did when he fought almost single-handed to have the polluted water system of Lowell changed to one of purity, and thereby saving many lives. This was his Victory of Peace.

"He was faithful to all the duties imposed upon him of whatever nature, and this bank will turn back its pages to his record and say with pride, 'Well done Thou Good and Faithful Servant'."

A life-long Unitarian, George Bowers devoted time and money to help his church. He was a member of the Boston Society of Civil Engineers, the New England Water Works Association, the Engineers Club of Boston, the Vesper Country Club, and a number of historical and philanthropical societies.

On October 24, 1878, Mr. Bowers was married to Estelle L. Wilkins, who survives him. He also leaves a daughter, Helen E. Bowers, and two sons, George W. Bowers, Assoc. M. Am. Soc. C. E., and Alton R. Bowers.

Mr. Bowers was elected a Member of the American Society of Civil Engineers on October 1, 1902.

STOERK JOHAN BRATAGER, M. Am. Soc. C. E.[1]

Died August 29, 1930

Stoerk Johan Bratager was born in Bergen, Norway, on June 7, 1860, a son of Lars and Cecelia (Lohn) Bratager, who came to Bergen from Voss, Norway.

He received his primary education in his native city, and was graduated from the Bergen Technical School in June, 1880. A short time later he left for Hanover, Germany, where he attended the Technical High School during 1881 and 1882. On his return to Norway he found the prospects for a position very poor. He continued his search, however, during the following winter, but finally decided to emigrate to the United States and arrived in New York, N. Y., in the spring of 1883.

From New York Mr. Bratager followed the route of his countrymen, finally reaching St. Paul, Minn. He quickly obtained temporary work with the St. Paul and Pacific Railway Company, now a part of the Northern Pacific System. As it happened, this "temporary job" in all its different manifestations continued for forty-two years, at the end of which time he retired at his own request because of ill health.

The positions held by Mr. Bratager during that period were, successively, as follows: 1883 to 1893, Draftsman and Chief Draftsman, the St. Paul and Pacific Railway Company; 1893 to 1898, Chief Draftsman; 1898 to 1901,

---

[1] Memoir prepared by Martin S. Grytbak, M. Am. Soc. C. E.

Assistant Engineer; 1901 and 1902, Division Engineer; 1902 to 1904, Office Engineer; January, 1904, to December 1922, Principal Assistant Engineer; and January 1923 to June 1, 1925, Assistant Chief Engineer, Northern Pacific Railway Company.

During his forty-two years of service Mr. Bratager was engaged in bringing the records and the physical condition of the railroad up to the high state of development which was required for taking care of the heavy traffic of 1925 as compared with the light traffic requirements of 1883. With a few assistants, he established the early distance schedules and records of the line, which, due to the rapid growth of the System, had become an important and difficult problem. Later, when valuation work started, he made the first complete valuation of the railroad.

While Mr. Bratager's service was generally devoted to the design of engineering undertakings and office administration, he was greatly interested in the application of these designs. He was in charge of the construction of fifteen miles of the Sykeston Branch during 1899. His design of the steel rollers and screws used in the moving of the east pier of Bismarck Bridge proved very successful. This pier, with load, weighed 4 700 tons, and was moved under traffic. On the completion of the work, it was found that 90% of the rollers had practically complete bearing after moving the pier 5 ft. easterly and 4 in. northerly, the rollers working between parallel planes.

During his long service with the Railway Company, Mr. Bratager gained the sympathy and confidence of his superiors, as well as the love and respect of his employees. All the engineers who received their early training under his direction will long remember him for his fair and correct dealings and his thorough investigation of the subjects at hand. His sympathy and help were of the greatest importance to all young engineers who, during his time with the Engineering Department of the Northern Pacific Railway Company, were looking for advice or a "start" with the Company.

Mr. Bratager disliked publicity. He was a friendly, quiet man of high principles and stern character, loved and respected by all who knew him. He enjoyed his work most of all, and his spare time was spent in working in his garden and around his home. In his earlier days he enjoyed traveling around in the "Twin Cities" and vicinity, alone, or in the company of a friend, generally winding up at some of the Company's track improvements or bridges. He was very interested in baseball and attended the games as long as he was able. He enjoyed the company of his family and friends, and they appreciated his keen humor and valued his friendship. He was a member of the Christ Lutheran Church of St. Paul.

On October 5, 1895, he was married to Synneva Bugge, the daughter of Mr. and Mrs. Ludvig Daae Bugge, of Valders, Norway. He is survived by his widow and his sister, Miss Carrie Bratager, of St. Paul, and one brother, Lars Bratager, of Minneapolis, Minn.

Mr. Bratager was elected a Member of the American Society of Civil Engineers, on June 3, 1915. He was also a member of the American Railway Engineering Association.

PETER FRANKLIN BRENDLINGER, M. Am. Soc. C. E.[1]

DIED DECEMBER 16, 1929

Peter Franklin Brendlinger was born on January 18, 1850, in New Hanover, Pa. He was the son of Frederick Brendlinger, a land owner and farmer, storekeeper, and Postmaster for many years, and, at one time, County Treasurer. His mother was Mary (Hill) Brendlinger who, like her husband, was of sturdy Pennsylvania German stock. His immediate forebears had never moved from this location since his great-grandfather had settled there in 1751, having emigrated from Getzigen in Wurtemburg, Germany.

Mr. Brendlinger prepared for a college education in the country schools of his home district near Pottstown, Pa., and at the Tremont Academy, at Norristown, Pa. He was graduated as a Civil Engineer in June, 1870, from the Polytechnic College of the State of Pennsylvania, in Philadelphia.

From August, 1870, to August, 1871, he was an Assistant Engineer on the location and construction of the St. Louis, Council Bluffs, and Omaha Railroad, from St. Louis, Mo., to Omaha, Nebr. Between October, 1871, and November, 1874, he served as Assistant Engineer on the Pittsburgh, Virginia, and Charleston Railroad, from Pittsburgh, up the Monongahela River, to Brownsville, Pa.; and in March and April, 1875, he was in charge of the Right-of-Way Department of the Allegheny Valley Railroad Company.

From April, 1875, to June, 1877, Mr. Brendlinger was Chief Engineer of Construction of the Point Bridge across the Monongahela River at Pittsburgh, surveys for which were started in 1875. The City Engineer of Allegheny, Pa., was the Consulting Engineer, and Mr. Brendlinger was Engineer in charge of the work, while the late Samuel Rea, Hon. M. Am. Soc. C. E., was an Assistant Engineer. The contract was awarded to the American Bridge Company, of Chicago, Ill. At that time the Point Bridge was the longest span chain bridge in the world. Designed by Edward Hemberle, an Austrian by birth, it had a span of 800 ft. In 1904, it was generally rebuilt and, in 1921, was replaced by a heavier and more modern structure.

From June until July 4, 1877, Mr. Brendlinger was in charge of testing American iron and steel for the United States Commission, headed by the late Q. A. Gillmore and Alexander L. Holley, Members, Am. Soc. C. E., and W. Leroy Smith. Between July 4, 1877, and November, 1878, he served as Assistant Engineer on the location of the Pittsburgh and Lake Erie Railroad from Pittsburgh to Youngstown, Ohio; later, he became Resident Engineer of Construction in charge of the Ohio River Division.

From November, 1878, to July, 1881, Mr. Brendlinger was Chief Civil Engineer for the Edgar Thompson Steel Works, at Braddock, Pa., which work included surveys for locating iron mines in Center County, with headquarters at Bellefonte, Pa. He resigned from the Steel Company in 1881 to become Assistant Chief Engineer of an important railroad project. In this connection, he was the recipient of a short autographed letter from Andrew Carnegie, reading in part as follows:

---

[1] Memoir prepared by Clark Dillenbeck and Charles S. Churchill, Members, Am. Soc. C. E.

"Cresson, Sept. 21, 1881

"MY DEAR MR. BRENDLINGER,

"Yours duly rec'd. It is always painful to me when one of the corps leaves the service but I can scarcely wonder that you longed to get back to Railroading.

"That is a great line you are building—a great property—I hope it will give Pittsburgh fair rates upon Coke. * * *

"With best wishes for your success,

"I am truly yours,
"ANDREW CARNEGIE."

From July 1, 1881, to January, 1884, Mr. Brendlinger was Assistant Chief Engineer for the Pittsburgh, McKeesport, and Youghiogheny Railroad during its location and construction from Pittsburgh to Connellsville, Pa., a distance of 60 miles. Between 1878 and 1881, the mileage of railroad construction in the United States was comparatively small, due to heavy mortgages and many receiverships on existing railroads. This undertaking in 1881 was the first of considerable moment in Western Pennsylvania. It was planned to be constructed on a relatively high standard as to roadbed, masonry, bridges, and track; and Mr. Brendlinger was selected to carry out these plans. He took great care in selecting the entire Engineering Department and energetically followed up every detail necessary to bring about the desired results. The line was opened for traffic on November 19, 1883, at a cost of about $5 000 000 and was in such good condition that, during 1884, it earned 6% interest on its cost, together with a surplus, and thereafter continued this good record.

From January 1, 1884, to April, 1888, Mr. Brendlinger was Chief Engineer of the Pennsylvania, Schuylkill Valley Railroad Company and of all Pennsylvania Railroad Lines between Hamburg and Nescopeck, Pa., on what is now the Sunbury Division of the Pennsylvania Railroad, on the North Branch of the Susquehanna River. Surveys were started on January 15, 1884, on that part of the line north of Hamburg, covering a section where there were difficult questions to be settled between the new line and both the Reading Railroad and the Schuylkill Canal; also, difficult questions arose from the necessity of crossing Broad Mountain. Many miles of preliminary lines were run, and topographical maps made, covering a large area between Hamburg and Hazleton, Pa., before the best route could be determined.

The final location, which included an approximately right-angle crossing of the outcrop of the Mamouth Coal Measure of the Mahanoy Valley, within the Stephen Girard Estate, where the coal has a thickness of 40 ft., involved the purchase of a pillar of coal as a support of the railroad with a depth of about 300 ft. The line was located in the spring of 1885 and contracts were awarded by Mr. Brendlinger in April of that year for a railroad with roadbed, masonry bridges, and track in accord with the standards of the Pennsylvania Railroad Company. The same characteristic expedition which he had practiced in Pittsburgh was used by Mr. Brendlinger, with the result that the line was completed and operated between Philadelphia and Pottsville, Pa., in November, 1886; and through to Nescopeck on the Pennsylvania Railroad, and thence, to Wilkes-Barre, Pa., in April, 1887, the new lines constructed north of Hamburg, covering 71 miles. Mr. Brendlinger received great credit for the completeness of the construction records and plans which were acclaimed as a great aid to the Operating Department of the Pennsylvania Railroad Company.

From April, 1888, to April, 1892, Mr. Brendlinger was General Superintendent of Construction for Brown, Howard, and Company of New York, N. Y., on the New Croton Aqueduct, with headquarters at Tarrytown, N. Y.

In 1892 he organized the firm of Brendlinger and Nearing, which was engaged in general contracting, doing work, until 1896, for the City of Yonkers, N. Y., for the Buffalo, Rochester and Pittsburgh Railroad Company, and for the Long Island Railroad Company. He also was associated with Pennell and O'Hearn, Contractors, on railroad construction for the Pennsylvania Railroad Company.

From 1896, until he retired from active business in 1914, Mr. Brendlinger was a General Contractor engaged on work for the Pennsylvania Railroad Company, the Pittsburgh, Cincinnati, Chicago, and St. Louis Railway Company, and the Philadelphia and Reading Railway Company, completing the following works: For the Pennsylvania Railroad Company, change of line, Pomeroy, Pa., and Oxford, Pa., at Milbourne Mills, League Island, and Lewistown Narrows, and on the Gallitzin Tunnels; for the Pittsburgh and Erie Railroad Company, Renovo to Keating, Pa.; for the Susquehanna Coal Company, at Millersburg, Pa.; for the Philadelphia and Reading Railroad Company, at Bowmansdale, Shippensburg, and Barnitz, Pa., and Hopewell, N. J.; for the Baltimore and Ohio Railroad, the Baltimore and Ohio Southwest Yard, at Philadelphia; for the Cleveland Short Line Railroad Company, at Cleveland, Ohio; and for the Pittsburgh, Cincinnati, Chicago, and St. Louis Railroad at Cambridge City, Ind.

Mr. Brendlinger was an easily approached, helpful man and a good friend. He exercised an uplifting influence on those with whom he was associated. His thoroughness and care in his engineering practice were, after 1888, followed in his contract work. The high standards of his entire life work were followed in all business and social dealings with others.

He died in Philadelphia, Pa., on December 16, 1929, from heart failure, caused by shock and exhaustion resulting from a fractured hip, due to a fall five weeks previous. He was within a month of completing his eightieth year, and prior to the accident had retained the keen and alert mental faculties which had always been his.

He was married to Hannah Emily Brown, on October 24, 1872, who died on April 27, 1924. There survive him four children: Margaret R. Brendlinger, George F. Brendlinger, William B. Brendlinger, and Mary B. Olmsted.

Mr. Brendlinger was elected a Member of the American Society of Civil Engineers on September 7, 1887.

ROBERT BROWN, M. Am. Soc. C. E.[1]

Died October 20, 1929

Robert Brown was born in Gatehouse of Fleet, Kirkcudbright Shire, Scotland, on September 23, 1860. He was educated at Borgue Academy and came to the United States in 1883.

Prior to his moving to this country and up until early in 1885, Mr. Brown was an Engineer in the British Army, engaged on surveying, bridging, submarine mining, and other engineering work. He served during the Egyptian War of 1882, for which he was awarded a medal and star for exceptional service in behalf of the British Government. He was then transferred to military engineering work in Bermuda, being assigned to fortification design.

From 1885 to 1917, he was engaged in engineering construction in many places throughout the United States, among which may be briefly noted the following: From 1885 to 1889, with Filbert and Porter, Philadelphia, Pa., on the construction of conduits and manholes; from 1889 to 1893, with the Bell Telephone Company, on conduit work; from 1893 to 1897 with the National Conduit Company, New York, N. Y., on a sub-contract for conduit and manholes; from 1897 to 1900 with Holbrook, Cabot, and Daley, on the Fourth Avenue Subway, New York, maintaining the Fourth Avenue track to 34th Street during construction, and building rubble stone from the roof of the tunnel and forming a conduit for the underground trolley; from 1900 to 1907 with Horgan and Slattery, Architects, as Inspector on the construction of the Hall of Records, the Tombs, an abattoir at 39th Street and Eleventh Avenue, the Butterick Building, the Harlem Hospital, the Irving National Bank, the First Battery Armory, as well as other buildings, in New York; during 1907, for eight months on expert insulation for a cold storage plant, at Cristobal, Canal Zone; and from 1907 to 1910, as sub-contractor on brick work for private houses.

In 1910 Mr. Brown went to Texas. Until 1915 he served with the American Construction Company, of Houston, in full charge of construction (acting as Engineer), for this Company on the Littlefield Building, at Austin; the Southwestern Life Building, at Dallas; the Union Belt and Terminal Station, at Houston; the Gulf, Colorado, and Santa Fé Station, the Nurses' Home and Women's Building ($3 000 000), and, from 1915 to 1917, on the reconstruction of the John Sealy Hospital, the State Medical College, and the Galveston Wharf Company's Piers 16 to 18 ($450 000), all at Galveston.

In 1917, he spent eight months as Superintendent of Construction, at Camp Travis, Division F, San Antonio, Tex.; in 1917 and 1918 he served as Construction Engineer for Stone and Webster, in France, on the erection of steel, ordnance machine shops, etc.; and from 1918 to 1921, he was Superintendent of Construction, for the Magnolia Petroleum Company, of a 28-story building, and on the design of other structures.

In March, 1922, Mr. Brown was commissioned as Major of Engineers of the United States Reserves, 8th Corps Area. He was a member of the City

---

[1] Memoir prepared by E. N. Noyes, M. Am. Soc. C. E.

Temple (Presbyterian Church), at Dallas, and served on its Board of Deacons. He was also actively interested in the construction and operation of the Reynolds Orphanage at Dallas.

Major Brown was a typical Scotchman in manner and speech and in the thoroughness with which he carried out the various projects with which he was connected. Starting in the building construction business in New York in 1885, when steel construction was in its infancy, and reinforced concrete was scarcely thought of, he saw and took part in the development of these materials, and during his life time handled them in some of the most important construction work in the United States.

He was always interested in matters pertaining to the Local Sections of the Society and made it a point to attend Section meetings whenever possible. He made friends easily and, being always extremely interested in the engineering work carried on by his friends in the profession, he took occasion to visit their projects and discuss details with them whenever possible. Major Brown was uniformly loved and esteemed by his engineer friends in Texas, and he will be missed greatly.

He is survived by his widow and a daughter, Scotia B. Brown, of Dallas, and by two sons, Preston W. and Robert C., of New York.

Major Brown was elected a Member of the American Society of Civil Engineers on November 21, 1921.

HENRY AMSDEN BURR, M. Am. Soc. C. E.[1]

DIED JUNE 20, 1930

Henry Amsden Burr, the son of Clarence and Blanche (Amsden) Burr, was born on December 11, 1890, in Granby, Que., Canada. Mr. Burr's parents were natives of Framingham, Mass., which was also their home, but were traveling in Canada when their son, Henry Amsden Burr, was born. The boy received his early education in Framingham. He enrolled in the Massachusetts Institute of Technology during 1909, and was graduated therefrom in 1913 with the degree of Bachelor of Science in Civil Engineering.

Following his graduation, Mr. Burr was employed by the Pennsylvania Steel Company at Steelton, Pa., as Draftsman and Structural Steel Detailer. He remained with this Company until April, 1914, when he was engaged by Stone and Webster, with offices at Cambridge, Mass., as Draftsman and Reinforced Concrete Detailer. This position was of short duration, due to a general reduction in forces in July, 1914.

He spent the remainder of 1914 and the first half of 1915 at his home in Massachusetts and in the West, most of this time being devoted to the study of his profession. He went to Nashville, Tenn., in August, 1915, and entered the employ of the Nashville, Chattanooga, and St. Louis Railway as Draftsman. During his connection with this Company, which lasted approximately five years, he served as both Draftsman and Designer.

---

[1] Memoir prepared by C. W. Butts, M. Am. Soc. C. E.

Mr. Burr resigned his position with the Nashville, Chattanooga, and St. Louis Railway in May, 1920, to enter the employ of the J. N. Chester Engineers on work in connection with the water supply of Nashville. He resigned this position, however, after about a month's service and accepted an appointment with the Tennessee Department of Highways and Public Works with headquarters at Nashville. He held this position from June, 1920, until June, 1930, during which time he served as Designer and Assistant Bridge Engineer. While he was connected with this Department, many bridges were designed and constructed, a number of which were large river crossings that were designated as Special Toll Bridges and authorized by Acts of the Legislature. These toll bridges span some of the larger streams of the State.

Mr. Burr's hobby, if it can be said that he had a hobby, was work. During his period of employment with the Department of Highways and Public Works he found time outside his regular working hours to design the structural steel work for the Nashville Trust Building and the Bennie Dillon Building, both of which are large office buildings in Nashville. Also, during this time, he was appointed by Davidson County, Tennessee, to check the plans and exercise general supervision of the construction of a bridge spanning the Cumberland River near Old Hickory, Tenn. All his work was done in a most satisfactory manner. Between 1915 and 1930, Mr. Burr found time to study commercial law and general business as well as more advanced work in engineering. He accomplished this through night school and correspondence schools.

Mr. Burr possessed a sterling character, indisputable integrity, and unquestioned reliability. He was an indefatigable worker, and had the confidence of his employers.

He was a Deacon of the Moore Memorial Presbyterian Church of Nashville, and was affiliated with the Masonic bodies, holding membership in Doric Lodge, F. and A. M., No. 732; Edgefield Chapter, No. 197, Royal Arch Masons; Nashville Council No. 1, Royal and Select Masters; Nashville Commandery No. 1; Trinity Consistory No. 2 (Thirty-second degree), and Al Menah Temple, A. A. O. N. M. S.; also, Doric Chapter No. 321, Eastern Star, all these lodges being in Nashville. He had served as Excellent High Priest of Edgefield Chapter No. 197, Royal Arch Masons; as Worthy Patron of Doric Chapter No. 321, Eastern Star, and, at the time of his death, was Junior Warden of Doric Lodge, F. and A. M., No. 732.

Mr. Burr was a member of the American Concrete Institute, the American Society for Testing Materials, and the Nashville Engineering Association, the last-mentioned body passing the following resolution, which also conveys the sentiment of his co-workers and acquaintances:

"*Whereas,* our valued friend and co-worker, Henry Amsden Burr, was lost through death, on June 20, 1930, to the membership of the Engineering Association of Nashville; be it

"*Resolved,* that this organization go on record as deeply deploring his untimely death and that we hereby give expression to our regard for his sterling qualities, both personal and professional.

"A native of Massachusetts and a graduate of Massachusetts Institute of Technology, Mr. Burr came to Tennessee in 1915, assisting at first in the design of structures on the N. C. & St. L. Railway, and later, with the design of many important bridges for the State Department of Highways. As a structural engineer and mathematician he was without a peer in our ranks. Recently, he did outstanding work on the committee appointed by the Association to prepare a revised draft of the Building Code of Nashville. Mr. Burr's career encompassed a multitude of useful engineering activities, all of which bore the stamp of excellence. As a friend and as a member, we shall miss him from our ranks.

"To his family, we extend our sincerest sympathies and transmit a copy of these resolutions."

"ENGINEERING ASSOCIATION OF NASHVILLE.

"Approved, October 6, 1930, by vote of the Association."

He was married on October 4, 1916, to Mrs. Margaret Schillinger, formerly Margaret Poyntz, who, with an adopted daughter, Charlotte Schillinger, and his two sons, Henry A., Jr., and John Ramsey Burr, survive him. He is also survived by his mother, Mrs. Blanche Amsden Burr, and two brothers, Clarence T. and Paul T. Burr, who reside in Framingham.

Mr. Burr was elected an Associate Member of the American Society of Civil Engineers on January 15, 1923, and a Member on October 10, 1927.

MAXWELL CUNNINGHAM BYERS, M. Am. Soc. C. E.[1]

DIED SEPTEMBER 23, 1930

Maxwell Cunningham Byers was born in Pittsburgh, Pa., on February 2, 1878. He was the son of John Morton and Caroline Mann (Lewis) Byers. His father, as Chief Engineer, located and constructed many hundreds of miles of the present railways of the United States. Several of his uncles were also similarly engaged, one having been Chief Engineer of the Reading Railway Company. His grandfather and granduncle were likewise railroad and, earlier, canal engineers, the former having been engaged with the Pennsylvania Railroad Company in laying out the first lines west over the Allegheny Mountains. His mother's father was a manufacturer engaged in the early iron industry. Her brother was also an Engineer and Railroad Executive, having been at one time President of one of the roads which is now a part of the Pennsylvania System and a Consulting Engineer in connection with the construction of the Eads Bridge across the Mississippi River, at St. Louis, Mo.

Mr. Byers was enrolled in the Civil Engineering Course in the Western University of Pennsylvania, at Pittsburgh, from 1894 to 1896. Finding it necessary to leave the University in 1896, at the end of his second year, he accepted a position in the Transportation Department of the Pennsylvania Railroad Company, and by study after working hours completed his course in mathematics and important engineering subjects.

---

[1] Memoir prepared by C. R. Gray, Pres., Union Pacific System, Omaha, Nebr.

His engineering experience began in 1898, as Assistant on the Engineer Corps of the Pennsylvania Railroad. Until 1899, he was in charge of grade revision on the Chicago Division, and, from 1899 to 1900, was engaged in the design and construction of masonry arches and the revision of grades on the Logansport Division. In 1900, he was promoted to the position of Assistant Engineer on design and construction of steel bridges and masonry on the Marietta Division, and, in 1901 and 1902, he had charge of general engineering on the Indianapolis Division, including grade revision, second-track work, design and construction of steel bridges, and masonry work. He attracted the attention of L. F. Loree, M. Am. Soc. C. E., then General Manager, who when he became President of the Baltimore and Ohio Railroad Company, took Mr. Byers with him to that property, where he served from 1902 to 1904 as Assistant Engineer, Maintenance of Way.

He transferred to the St. Louis-San Francisco Railway Company with Mr. Loree in 1904, and was, successively, Assistant Engineer, Maintenance of Way; Engineer, Maintenance of Way, Third District, in charge of the Engineering Department, including the design and construction of bridges and buildings and also of river protection work on the Mississippi River; Assistant to the General Manager in charge of all construction on the System; Engineer, Maintenance of Way, in charge of the Engineering Department; and, from 1909 to 1913, he was first Chief Engineer and then Chief Engineer-Operation, his service being continuous with the St. Louis-San Francisco Railway Company from 1904 to 1913.

Mr. Byers was appointed Assistant to the President of the Great Northern Railway Company in 1913, a position which he held until March, 1914, when he entered the service of the Western Maryland Railway Company, which he served continuously until his death. From 1914 to 1918, he was Assistant to the President. During the period of Federal Control—1918 to 1920—he was General Manager and Federal Manager, with his jurisdiction extended over the Cumberland Valley Railway and the Cumberland and Pennsylvania Railroad Companies. From March 1, 1920, until his death, he was President of the Western Maryland Railway Company, and from 1926 had also served as Chairman of the Board of Directors.

Mr. Byers was a member of the Duquesne Club of Pittsburgh, and the Maryland Club and the Merchants Club of Baltimore, Md. He was a Director and Member of the Executive Committee of the Union Trust Company of Baltimore; a Director of the Eutaw Savings Bank, the First National Bank of Baltimore, and the Hagerstown Bank and Trust Company; a corporate member of the Hopkins Place Savings Bank of Baltimore; a Director of the Baltimore Association of Commerce; and a Director and Vice-President of the Industrial Corporation of Baltimore. Mr. Byers also belonged to the American Railway Engineering Association, the American Railway Association, and the American Railway Guild.

He had been a member of the Catonsville, Md., Presbyterian Church since 1914, serving most of the time as a member of the Board of Trustees. He was one of the most interested of the Building Committee during the development of the new church property at Catonsville, a suburb of Baltimore.

Mr. Byers was married at Point Pleasant, W. Va., on November 28, 1906, to Janet McCulloch who, with five sons, John Maxwell, Lewis Cunningham, Richard McCulloch, James Alfred, and Robert Morton, as well as a sister, Mrs. Mary E. B. Barbee, and a brother, Morton Lewis Byers, M. Am. Soc. C. E., survives him.

From 1904 to 1920, he was associated with the writer, and after the business relationship ceased, a close and intimate acquaintance was maintained. Mr. Byers was first an Engineer and an exceedingly good technician. His mind was an intensely practical one, and he was an ardent student of railroad operation, invariably working out problems with a view to producing practical economies in the movement of traffic. While he was still in the Engineering Department his advice was always eagerly sought by the operating officers, and their confidence in him was conspicuously shown, but it was not until he became President in 1920 that the real opportunity came to put into effect his own ideas of operating a railroad. The results speak for themselves. While steadily improving the physical condition of the roadway and equipment, he produced a transportation ratio which attracted wide attention and, as is well known, it is this feature of the operating side wherein efficiency is best reflected.

Mr. Byers was only 52 years of age when he died, and at the height of his intellectual powers. He was striking in appearance, his dark eyes sparkling with animation, and he had a most engaging personality. His work and his family engrossed his time. He had no fads and played little, but he had a capacity for work that was astounding, and as is natural this communicated itself to his entire organization. He was a bright, fearless, and honest man, a true friend, and an outstanding executive and railroad operator.

Mr. Byers was elected a Member of the American Society of Civil Engineers on June 6, 1911.

WILLIAM CAIN, M. Am. Soc. C. E.[1]

Died December 6, 1930

William Cain was born at Hillsboro, N. C., on May 14, 1847, the son of Dr. William and Sara J. (Bailey) Cain, of English, Scotch, Irish, and Welsh descent. His early education was received in a private school in Hillsboro, and at the Hillsboro Military Academy. In 1866, he received the degree of Master of Arts from the North Carolina Military Polytechnic Institute. The military training he acquired in these institutions enabled him as a lad of fourteen to drill troops for the Confederacy, a service which he rendered with conspicuous success.

Mr. Cain then became Assistant Engineer to W. C. Kerr, State Geologist, in the preparation of a map of North Carolina. Afterward, for a period of about eight years, he engaged actively in railroad engineering, as follows: 1867, Assistant Engineer, Western North Carolina Railroad Company; 1868,

---

[1] Memoir prepared by T. F. Hickerson, M. Am. Soc. C. E.

Assistant Engineer, on the line from Greensboro to Salem, N. C.; 1868 to 1874, Assistant Engineer and then Chief of Party on surveys and locations, also Resident Engineer, Raleigh and Augusta Air Line Railroad Company; Cape Fear and Yadkin Valley Railroad Company; and Wilmington, Columbia, and Augusta Railroad Company. From 1874 to 1880, he was Professor of Mathematics and Engineering at Carolina Military Institute, Charlotte, N. C. From 1880 to 1882, he was again engaged in the practice of railroad surveying and engineering as Chief of Party on the following projects: Lines from Lenoir, N. C., *via* Cook's Gap on the Blue Ridge, to Elizabethton, Tenn.; the Midland Railroad, from Salisbury, *via* Asheboro, to Goldsboro, N. C.; and the Atlantic Coast Line Railroad, from Wilson, N. C., *via* Fayetteville, to Pedee, S. C.

In 1882, he was elected Professor of Mathematics and Engineering at "The Citadel, the Military College of South Carolina", Charleston, S. C. In 1888, he became Professor of Mathematics and Engineering at the University of North Carolina, Chapel Hill, N. C., which position he held until his retirement from active teaching in 1920, with an adequate pension in recognition of his long and distinguished service. He then became Professor Emeritus, but only in name, for his labors never ceased. His brilliant and painstaking articles and his constructive discussions of current papers continued to appear none the less frequently in the *Proceedings* of the Society. On the very day of his death, at the age of 83, when he was struck the fatal blow by an automobile while attempting to cross the street in front of his home, manuscripts dealing with the theory of dams—sent to him for criticism by prominent engineers in practice—were found on his desk.

A veteran of the Civil War, Professor Cain brought forward into this modern age the bearing and manners, the ideals, the loyalties of the old South. And yet, with all his treasured memories and his alliance with the past, he kept abreast of the times. To the last he maintained a lively interest in the drama of passing events—in the changing aspects of science and literature, in invention, in the affairs of Government, in fashion, and in sports.

His chief recreations were hunting, fishing, and violin music, all of which had to be given up during the last two years of his life on account of injuries sustained in a previous automobile accident.

Professor Cain was one of the original five Kenan Professors, who were chosen as a mark of distinction by the Faculty of the University of North Carolina in 1918. The thoroughness and accuracy of his character marked his class-room teaching, and his distinguished scientific contributions to Engineering redounded to the credit of his institution. He received honorary degrees of Doctor of Laws from the University of South Carolina (1916), and Doctor of Science from the University of North Carolina (1920).

He was a member of the American Mathematical Society, and a Fellow of the American Association for the Advancement of Science. Professor Cain was a member of the Protestant Episcopal Church, and gave generously to its support. He served on the Vestry of the Chapel of the Cross a number of times, and his clear mind, coupled with his sense of humor, was always helpful.

In 1923, the Society presented him with the J. James R. Croes Medal for his paper, "The Circular Arch Under Normal Loads".[2]

Professor Cain's scientific publications date from 1874, when his paper, "Uniform Cross-Section and T Abutments", appeared in the November issue of *Van Nostrand's Engineering Magazine.* In his paper on "Earth Pressure",[3] he showed that all the experiments given by the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., in his notable paper on "The Actual Lateral Pressure of Earthwork", at least in cases where the backing was sand, nearly devoid of cohesion, could be reconciled with theory by rejecting the "ancient" theory used by Baker and adopting the "wall-friction" theory in which the friction of the earth on the wall is considered. In his paper, "Trusses with Superfluous Members",[4] there was presented for the first time to American readers the theory of trusses with redundant members. In "Positions of Live Loads Giving Maximum Strains for Single-Intersection Trusses",[5] a thorough discussion of this subject, about which little had been written, was given; and the method used in this paper furnished the basis for that given in the Second Edition of Professor Cain's book entitled "Maximum Stresses in Framed Bridges".

Another important contribution to engineering practice was the development, in a paper entitled "Unit Stresses",[6] of the "Cain-Launhart formula", in which Professor Cain modified Launhart's formula to allow for impact. Other valuable papers written by Professor Cain at this time deal constructively with the following subjects: Long Columns, High Masonry Dams, and Portal Bracing.

In 1891, in his "Determination of the Stresses in Elastic Systems by the Method of Least Work",[7] Professor Cain first introduced to American engineers the method of Castigliano, as embodied in his "Systémes Elastiques" (Turin, 1879). Professor Cain herein gave original demonstrations of the theorem of least work for framed structures with redundant members, by use of the method of virtual work, with applications.

In "Stresses in Masonry Dams",[8] Professor Cain presented a novel method for the determination of stresses at the battered faces of high masonry dams. In 1911, he published an elaborate paper, "Experiments on Retaining Walls and Pressures on Tunnels",[9] in which for the first time cohesion in earth is included, to reconcile the carefully conducted experiments of Leygue on small models, with theory; also, a complete graphical treatment of pressures in earth endowed with both friction and cohesion is given for the first time.

Following the publication of a paper on "A Shortened Method in Arch Computation",[10] by H. A. Sewell, M. Am. Soc. C. E., Professor Cain published one month later the demonstration of a still shorter method.[11] In 1914-15, he presented two exhaustive papers, "Stresses in Wedge-Shaped Reinforced Concrete Beams",[12] and "Cohesion in Earth: The Need for Comprehensive Experimentation to Determine the Coefficients of Cohesion",[13] in the latter of which all experiments available are given and shown to agree with Coulomb's hypothesis. The need for extensive experimentation is herein pointed out and apparatus in view criticized.

---

[2] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

[3] *Van Nostrand's Engineering Magazine*, February, 1882.

[4] *Loc. cit.*, October, 1882.

[5] *Loc. cit.*, March, 1885.

[6] *Engineering News*, 1887.

[7] *Transactions*, Am. Soc. C. E., Vol. XXIV (April, 1891), p. 265.

[8] *Loc. cit.*, Vol. LXIV (1909), p. 208.

[9] *Loc. cit.*, Vol. LXXII (1911), p. 403.

[10] *Loc. cit.*, Vol. LXXVI (1913), p. 133.

The following papers and monographs by Professor Cain,[14] dealing with mathematical and engineering problems, are given only by title: "Sanitary Engineering," Third Edition, North Carolina Board of Health, 1885; "Brief Demonstration of the Method of Least Work,";[15] "The Transition Curve,";[16] "On the Fundamental Principles of the Differential Calculus";[17] *Road Bulletin,* North Carolina Geological Survey, 1893; "The Stone Arch,";[18] "Exact Computation of Stresses in a Queen-Post Truss,";[19] "A New Method of Laying Out Grades for Sewers by Aid of the Transit";[20] "Note on the Algebraic Form, $\frac{0}{0}$";[21] "Imaginary Roots of a Cubic";[22] "Grain Pressures in Deep Bins";[23] "Crown Thickness of Masonry Arches"[24]; and "The Teaching of Elementary Dynamics".[25] A number of articles on arches, stresses in bridges, retaining walls, algebra and geometry, appeared in *Van Nostrand's Engineering Magazine,* and these were at once published in book form in Van Nostrand's Science Series, with the following original titles: No. 12, "A Practical Theory of Voussoir Arches" (1874); No. 38, "Maximum Stresses in Framed Bridges," (1878); No. 42, "Voussoir Arches Applied to Stone Bridges, Tunnels, Domes, and Groined Arches" (1879); No. 48, "Theory of Solid and Braced Elastic Arches" (1879); and No. 73, "Symbolic Algebra and Notes on Geometry" (1884). In 1888, at Mr. Van Nostrand's request, Professor Cain revised Arthur Jacob's "Practical Designing of Retaining Walls," doing it so thoroughly that he retained from Jacob's book only one figure. Eventually, Jacob's name as author was displaced by that of Professor Cain, and the book has passed through many editions.

In the First Edition of Science Series No. 42, (1879), an advance is made in the treatment of the stone bridge by considering the elasticity of the material, but the theory of the "elastic arch" is not yet applied in a precise manner. This volume contains much new material on the "barrel" or common arch, on tunnel arches, groined and cloistered arches, and on domes. A new edition (1901), bore the title, "Theory of Steel-Concrete Arches and of Vaulted Structures." Herein was given, for the first time, a complete, practical method for dealing with the reinforced arch of variable section by the elastic theory. The two books on reinforced arches, Science Series Nos. 42 and 48, have been repeatedly cited as authoritative in technical journals, and the methods set forth therein have been used in the design of a number of bridges. The distinguished English engineer, James Marsh, paid Professor Cain a high compliment by publishing his arch theory, with Professor Cain's permission, in his extended work on "Reinforced Concrete" (London, 1904).

---

[11] *Transactions,* Am. Soc. C. E., Vol. LXXVI (1913), p. 138.

[12] *Loc. cit.,* Vol. LXXVII (1914), p. 745, and Vol. LXXVIII (1915), p. 740.

[13] *Loc. cit.,* Vol. LXXX (1916), p. 1315.

[14] *Journal* of Elisha Mitchell Soc., Vol. XXXIX, 1924, Univ. of North Carolina.

[15] *Engineering Magazine and Science Journal,* Vol. VIII, 1891.

[16] *Transactions,* Am. Soc. C. E., Vol. XXVI (1892), p. 473.

[17] *Engineering Magazine and Science Journal,* Vol. IX, Pt. I, 1892.

[18] *Loc. cit.,* Vol. X, 1893.

[19] *Loc. cit.,* Vol. XIII, Pt. II, 1896.

[20] *Loc. cit.,* 1901.

[21] *Loc. cit.,* Vol. XIX, Pt. I, 1903.

[22] *Loc. cit.,* Vol. XIX, Pt. II, 1903.

[23] *Engineering News,* May 12, 1904.

[24] *Loc. cit.,* October and November 17, 1904.

[25] *Science,* July 7, 1916.

A useful textbook, written with Professor Cain's habitual lucidity, is a "Brief Course in the Calculus", (1905). The results of many of his published papers and a wealth of additional research are incorporated in his "Earth Pressure, Retaining Walls and Bins" (1916).

One of Professor Cain's most valuable contributions to engineering literature was his paper previously mentioned on, "The Circular Arch under Normal Loads," in which he developed, by refined mathematical methods, formulas for computing moments and thrusts, and the resulting stresses in arches with "fixed" and "hinged" ends. This paper marked a distinctly forward step in advance of the approximate solutions then in vogue. Subsequently, Professor Cain, in a discussion[26] of a paper by B. F. Jakobsen, M. Am. Soc. C. E., entitled "Stresses in Thick Arches of Dams", developed further formulas, using more rigorous methods in certain particulars. The Cain formulas soon became established as acceptable standards in practice, as well as theory, by the Federal Power Commissions and by State and local public officials as well.

Professor Cain discussed[27] the elaborate paper by Cecil Vivian von Abo, Assoc. M. Am. Soc. C. E., on "Secondary Stresses in Bridges". As stated, he also gave a long and able discussion[26] of Mr. Jakobsen's paper on "Stresses in Thick Arches of Dams", in which frequent comparisons and references to the Cain formulas are cited, and acknowledgment is made to Professor Cain for encouraging its submission to the profession. Among the subsequent papers to which discussions were contributed by Professor Cain are the following: "Experimental Deformation of a Cylindrical Arched Dam",[28] by B. A. Smith, M. Am. Soc. C. E.; "Analytical Solution of Masonry Domes",[29] by David C. Coyle, M. Am. Soc. C. E.; "Design of Symmetrical Concrete Arches",[30] by Charles S. Whitney, M. Am. Soc. C. E.; "Notes on Arched Gravity Dams",[31] by B. F. Jakobsen, M. Am. Soc. C. E.; "A Graphic Method for Determining the Stresses in Circular Arches under Normal Loads by the Cain Formulas",[32] by Frederick H. Fowler, M. Am. Soc. C. E.; "Analysis of Arch Dams by the Trial Load Method",[33] by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq.; "Load Distribution in High Arch Dams",[34] by R. A. Sutherland, Assoc. M. Am. Soc. C. E.; "Laminated Arch Dams with Forked Abutments",[35] by Fred A. Noetzli, M. Am. Soc. C. E.; and "Stresses in Gravity Dams by Principle of Least Work",[36] by B. F. Jakobsen, M. Am. Soc. C. E.

---

[26] *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 522.

[27] *Loc. cit.*, Vol. 89 (1926), p. 151.

[28] *Loc. cit.*, Vol. 91 (1927), p. 712.

[29] *Loc. cit.*, Vol. 88 (1925), p. 119.

[30] *Loc. cit.*, Vol. 88 (1925), p. 1030.

[31] *Loc. cit.*, Vol. 92 (1928), p. 795.

[32] *Loc. cit.*, Vol. 92 (1928), p. 1550.

[33] *Loc. cit.*, Vol. 93 (1929), p. 1265.

The first draft of Mr. Jakobsen's paper on "Stresses in Gravity Dams by Principle of Least Work", was submitted in advance for criticism to Professor Cain; and, consequently, his discussion was published simultaneously with the paper. It is singular that this unusual procedure occurred so shortly before Professor Cain's death, thus bringing to a glorious end a long series of constructive scientific articles covering a period of fifty-seven years.

The Student Chapter of the Society at the University of North Carolina was named "William Cain Civil Engineering Society", in his honor. Those who knew him well, called him "Major" Cain, a title he has held since his early days while teaching at a military school. He never was married. During recent years, he was a member of an "eating club" in Mrs. Patterson's home just across the street from where he lived. It was while attempting to cross the street to join his friends at the table that he met his death. The members of the club consisted of a dozen or so bachelors from the University Faculty, and they were called the "Old Soldiers". The following sentiment by one of them was published in *The Chapel Hill Weekly*:

> "As Major Cain with unconscious strength held on to life after those terrific blows upon head and body, he was, in those last dozen hours, making death a true part of his valiant life. He had refused to surrender to age or deafness or any circumstance. In his eighty-third year he did his day's work with the zest of a great engineer who with the higher mathematics of strains and stresses built bridges and dams against both time and power to the end.
>
> "One morning I found him in his room with papers all about him—an engineering treatise from Australia for review, a mathematical article from England for critical calculations, and the set-up of a dam in California in need of testing by his engineering formulas. In the quiet of his simple room, roads, dams, and bridges projected from his mind vigorous and clear over all.
>
> "Not only so, but in that lovely home where Miss Blanche made gay among her 'Old Soldiers', the Major was the gayest of them all. With all his fun and kindly insight, his was a life of fierce loyalties and deep indignations. Injustice and shams blenched before his utter scorn. The 'Old Soldiers' loved the gracious strength of him, forthright first and gracious always. In his handsome person, tall and unbending, the spirit of the Old South lived on with unconscious courtliness into the midst of the new, bidding us to pause in our raucous hurry.
>
> "He had been asked to retire to the shelter of relatives and take his ease, but he chose to do his day's work of high thinking in his own quarters. He scorned to let people guide him across the street even in darkness and mists. He met life face forward and death head on. His was the most gallant figure that walked our village streets."

Professor Cain was elected a Member of the American Society of Civil Engineers on November 7, 1888. He served as a Director of the Society from 1912 to 1914.

---

[34] *Transactions,* Am. Soc. C. E., Vol. 93 (1929), p. 1666.

[35] *Loc. cit.,* Vol. 95 (1931), p. 566.

[36] *Proceedings,* Am. Soc. C. E., September, 1930, Papers and Discussions, p. 1639.

GEORGE LINDEN CAMPEN, M. Am. Soc. C. E.[1]

Died September 21, 1930

George Linden Campen, the son of John L. and Ann Hartford Campen, was born at Pittsford, Mich., on July 27, 1867. His preliminary education was obtained in the grade school at Pittsford and the High School at Hillsdale, Mich. He taught a number of terms in the district schools in order to help provide the means for his attendance at the Normal College at Ypsilanti, Mich., from which institution he was graduated in the Class of 1885.

Owing to his father's health, his parents moved to Wahoo, Nebr., in 1884, where Mr. Campen joined them after his graduation from the Michigan State Normal College. In 1885, he became an Instructor in the Wahoo grade schools, teaching through the years of 1885 and 1886 and until the spring of 1887, when he resigned to become a Rodman with the Burlington and Missouri River Railroad Company.

Deciding on an engineering career, Mr. Campen enrolled with the University of Nebraska, at Lincoln, in the fall of 1887, as a Special Student in Engineering, and completed his course in the spring of 1890. During the summer months of his post-graduate years he was employed as Rodman by the Burlington and Missouri River Railroad Company and as Rodman and Computer in the City Engineer's Office at Lincoln.

After completing his post-graduate course, his first position was with Kilpatrick Brothers and Collins, Cambria, Wyo., as Mine Surveyor which position he held for about six months. He left that firm to make a topographical survey for the Sundance Coal Company, in Wyoming.

In October, 1891, Mr. Campen entered the City Engineer's Office at Lincoln, as Assistant Engineer of Construction, in charge of the construction of about twenty miles of sewer laterals. In April, 1896, he was promoted to the position of Deputy City Engineer in charge of the construction and repair of sewers and paving, but resigned in 1898 to accept a position as Construction Engineer with the firm of J. R. Sheely and Company, Bridge and Municipal Contractors.

With the J. R. Sheely and Company, his work consisted of designing, bidding on projects, and the supervision of construction of municipal work and bridges. He resigned this position in July, 1901, to accept the appointment as Deputy Engineer in the City Engineer's Office at Lincoln; the same month he was appointed City Engineer, responsible for the maintenance and construction of water and sewer mains, pavements, pumping, and municipal lighting plants. He also was in charge of the construction of three bridges over the Loup River in Nance County, Nebraska, and a concrete dam across the Blue River at Fairbury, Nebr.

In December, 1905, by special leave from the City of Lincoln, Mr. Campen joined the Government forces at the Canal Zone, Panama. He resigned as City Engineer four months later, and assumed his duties as Water Commissioner in direct charge of preparing and carrying into effect plans and rules for the sanitary installation of plumbing, use of water, maintenance of service connections, and protection of paving in Panama and Colon; in December, 1906, he was made Superintendent of Public Works in the Canal Zone. In this position he recommended public improvements, supervised and directed inspectors of plumbing, the construction and maintenance of water mains, sewerage systems, and paving in Panama and Colon.

---

[1] Memoir prepared by E. L. Brown, M. Am. Soc. C. E.

Mr. Campen was married in 1892 to Pearl Hershey, at her home at Hillsdale, Mich. He had met her at Hillsdale while attending High School there. Mrs. Campen accompanied her husband to the Canal Zone.

While he was employed in the Canal Zone he was offered a similar position with the Cuban Government. This position he declined at the request of the Governor-General, but a few months later, in 1909, he was compelled to resign his position and return to the United States because of his wife's ill health. Mrs. Campen died three years later and was laid to rest in her childhood home, Hillsdale.

In August, 1909, Mr. Campen accepted the position of Assistant City Engineer of Omaha, Nebr., under the City Engineer, G. W. Craig, Assoc. M. Am. Soc. C. E. In this capacity he was in charge of the construction and maintenance of pavements, sewers, and sidewalks. He resigned his position with the City of Omaha, April 30, 1912, and entered private practice as a Consulting Engineer with an office in Omaha where he remained until March 1, 1914. He then organized the Campen Company, of Omaha, as Contracting Engineers on foundation work, and became its President; because of financial difficulties the Company was dissolved in January, 1916.

He again entered the contracting business in July, 1916, when he became Superintendent of Construction for the Haynes Elevator Company, of Omaha, on a 1 000 000-bushel concrete elevator. This elevator was completed in July, 1917. He then accepted a similar position with W. E. Wickham, of Omaha, on the construction of the cantonment at Fort Omaha. He remained with this Company only three months, having received an appointment as Senior Highway Engineer with the U. S. Bureau of Public Roads, with headquarters at Omaha. This work consisted of the location, construction, and maintenance of Federal Aid Highways in the State of Iowa. He held this position until his death on September 21, 1930.

In May, 1917, Mr. Campen was married a second time to Therese Goodwin, of Omaha. To this union was born a son; but, again, Mr. Campen was left a widower, for, after nine years of happy married life, his wife passed away in February, 1926, from influenza.

For forty years Mr. Campen practiced as a Civil Engineer, most of which time he was a public servant, serving the City and Federal Government with sterling integrity and ability, his work having outlived him. He constructed the first asphalt pavement laid in the City of Lincoln, and pavement laid by him years ago in Omaha is in excellent shape to-day.

He was a strong character, fearlessly honest, a dynamic personality. This gained him many friends and he was respected and trusted by all. He took a keen interest in his work and always was ready to try an experiment for the best results and as quickly to discard it when the occasion required it. Mr.

Campen was a keen, far-sighted engineer, willing to give his opinion when requested. There are a number who regret that they did not follow the advice given by him; and many an engineer, reviewing his past services, can appreciate the kindly help and advice given to him by Mr. Campen; for this reason some of his friends called him the "Dean of Highway Engineers".

He had been failing in health for about a year and, on Sunday, September 21, 1930, after placing a floral wreath on the grave of his wife, accompanied by his mother-in-law and a friend, he died at the wheel of his car while returning to his home. He is survived by his son, George L., his father and mother at Pittsford, a brother, E. J. Campen, of Denver, Colo., and an adopted son, William, President of the Omaha Testing Laboratories. The funeral service was held in the Scottish Rite Temple, and he was laid to rest in Forest Lawn Cemetery, at Omaha, beside his wife.

Mr. Campen was elected a Member of the American Society of Civil Engineers on October 5, 1909.

WILLIAM WALLER CARSON, M. Am. Soc. C. E.[1]

DIED FEBRUARY 7, 1930

William Waller Carson was born on June 2, 1845, in Adams County, Mississippi. Both his father and mother were descended from a long line of leaders who from Colonial times had been active in the development of the South.

Soon after the birth of William, the family moved to a plantation in Louisiana on the banks of the Mississippi River, about 35 miles above Vicksburg, Miss., and here his boyhood was spent in the surroundings of Southern hospitality and plantation life. Hunting, swimming, fishing, horseback riding, the home teaching, first by his mother and later by tutors—all these contributed to the physical vigor and moral character which were his through a long and active life. In a home the head of which was responsible for the physical health and moral training of more than 500 human beings, he learned that superior position brings responsibility, and while he was still a boy he appreciated that responsibility and carried his full share.

This free and independent life was interrupted suddenly by the Civil War. The Mississippi River became the path for gunboats. Flashes from cannon near Vicksburg lighted the sky. Although he was too young to enlist when the war broke out, Mr. Carson entered the Confederate Army at the age of 18, and served through the remaining two years of the war, first as Private and, later, as Sergeant-Major of the 4th Louisiana Cavalry.

These two years, filled with tedious work, exciting adventure, deprivation, and suffering, printed upon his memory many pictures which remained bright and vivid to the end of his life. He received a wound which nearly proved fatal, and which interfered seriously with his future work. Due to bad drinking water, he suffered for forty-six years from a condition under which a less vigorous constitution would not have survived.

---

[1] Memoir prepared by Dr. Charles A. Perkins, Knoxville, Tenn.

At the close of the war, like many other young men, Mr. Carson followed General Lee to Washington College—now known as Washington and Lee University—where for four years he studied to prepare himself for work as a civil engineer.

A most urgent need in the South at this time was the construction of highways, railroads, and public works, and, after receiving the degree of Civil Engineer (1868) and Mechanical Engineer (1869), he entered the office of the New Albany and St. Louis Railroad Company and, later, accepted similar work in the Sewer Department of the City of Louisville, Ky. From 1870 to 1873, he was employed on the Southern Division of the Mobile and Grand Trunk Railroad, occupying, successively, the positions of Transitman, Resident Engineer, and Engineer in Charge of Maintenance of Way of the completed Division.

In 1874, Mr. Carson went to New York, N. Y., and attended the School of Mines, Columbia University, for one session. From this time, his principal attention was given to his main life work—that of teaching. For six years, from 1877 to 1883, he was Professor of Mathematics at Davidson College, and, in 1885, he went to the University of Tennessee, in Knoxville, Tenn., where he spent the remainder of his life. At this time there were less than 200 students in the University, and of the 13 men on the Faculty, Professor Carson was the only one with engineering training or experience. Two years later—in 1887—there was a complete re-organization of the University, and of the former Faculty he was the only member to be retained by the new Administration. The standing almost immediately accorded to this institution in educational circles bears witness to the scholarship and teaching ability of the men who formed the new Faculty.

As a teacher, Professor Carson spared no effort to inculcate true engineering ideals in his students. He had too much practical experience to tolerate any slovenly or inaccurate work. His mind was too severely logical to overlook careless thinking or expression. Yet his boundless good nature, his kindness, and his warm heart, attracted his students and made them his lifelong friends. Entirely sincere and absolutely upright in every act and thought, his fine character could not fail to command respect and impress itself upon all who came under his influence.

During his years as teacher, Professor Carson found time to write articles on a number of engineering subjects: Stresses in arches, transition curves in railway location, experiments on road resistance with different types of surfaces, articles on the calculus, etc. He designed and patented a method for protecting the banks of the Mississippi River from erosion, based on his familiarity with that stream. (The plantation where he spent his boyhood had long since been carried away by this river.)

Professor Carson interested himself in the welfare of the men who had fought by his side, and, as Commander of the Fred Ault Camp of the United Confederate Veterans for the last eighteen years of his life, he spent much of his time in making their conditions of living more comfortable. The monument which he has left is in the lives of those whom he taught and influenced.

In 1880, he was married to Rachel Finnie, of Memphis, Tenn., who survives him. Also surviving him are three children, Katherine W., Emma Carson McIlwaine, and W. Waller, Jr. One son, J. Finnie, died while he was a student at the University of Tennessee.

Professor Carson was elected a Member of the American Society of Civil Engineers on November 2, 1892.

MAURICE WURTZ COOLEY, M. Am. Soc. C. E.[1]

Died April 27, 1930

Maurice Wurtz Cooley, the son of Justus and Elizabeth (Pine) Cooley, was born at Little Britain, N. Y., on August 16, 1861. He was directly descended from a Colonial great-great-grandfather who was the first Governor of the Free People of Dover, N. H., and, in a following generation, from those who served in the Revolutionary War.

Mr. Cooley received his preliminary education at Phillips Exeter Academy at Exeter, N. H., from which he was graduated in 1883. He then entered the Massachusetts Institute of Technology, where he was a student in Civil Engineering from 1884 to 1886, his summer vacations being spent as Rodman and Leveler for the late A. H. French, M. Am. Soc. C. E., of Brookline, Mass.

On leaving college, Mr. Cooley was engaged in the following positions: From July, 1886, to October, 1887, Draftsman and Instrumentman, for the Toledo, St. Louis and Kansas City Railroad Company; 1887 to 1889, Instrumentman, the Lake Shore and Michigan Southern Railway Company; 1889 to 1892, Assistant Engineer, the Michigan Southern Division of the Lake Shore and Michigan Southern Railway Company, in charge of drafting-room and field parties; the making of designs for bridge masonry, large terminal yards, and also for many changes of line and grade, including 150 miles of second-track construction; 1892 to 1898, and from February to October, 1899, Engineer, Maintenance of Way, the Toledo and Ohio Central Railway Company, in charge of the maintenance of bridges, buildings, and roadway.

During the Spanish-American War Mr. Cooley served as First Lieutenant in command of Company B, 2d U. S. Volunteer Engineers, from July, 1898, to February, 1899, designing and constructing a plant for supplying the 7th Army Corps with water near Havana, Cuba; he was also engaged on the construction of a dock for landing troops at the mouth of the Almendares River, and, in addition, he made surveys and designed a dock at Matanzas, Cuba, for which services he was promoted to the rank of Captain of Engineers.

After the war work was finished in Cuba, Captain Cooley served from November, 1899, to May, 1900, as Assistant Engineer of Construction on the Baltimore and Ohio Railroad in charge of the construction of a coal pier at Curtis Bay, Md., and the design of a terminal yard and masonry for the reconstruction of the Curtis Bay Branch. From May, 1900, to 1903, he was Manager of the National Contracting Company, having charge of the Company's business projects which consisted of the construction of a dam at Glens Falls, N. Y., and several sections of the Metropolitan High Level Sewer at Boston, Mass. From 1903 to 1906, he served as Manager of the New York and Bermudez Company and General Agent for the Trinidad Lake Asphalt Company, of Trinidad, British West Indies; and from 1906 until 1921, as General Manager of the Gilson Asphaltum Company and the Uintah Railway Company. He constructed the Uintah Railway, which is said to be the most crooked line in the United States. It utilizes 70 miles of trackage to traverse 40 miles in the mountains of Eastern Utah. It has several 3 and 4% grades and one 66° curve. This line is used to haul the gilsonite ore to the Denver and Rio Grande Western Railway at Mack, Colo.

---

[1] Memoir compiled from information on file at the Headquarters of the Society.

In 1922 Captain Cooley entered the service of the American Asphalt Company in St. Louis, Mo., and, in 1924, went to Cuba to open mines for the Central Commercial Company, of Chicago, Ill., which had large interests there. During his last few years he lived in New York, N. Y., where he was actively interested in real estate promotion and in the sale of securities.

In April, 1930, Captain Cooley was summoned to Salt Lake City, Utah, as a witness in a suit between the Gilsonite Asphaltum Company and the Utah Gilsonite Company. While on this trip he stopped over a few days to visit with friends at Mack, Colo., where he was suddenly taken ill, and died in a hospital at Grand Junction, Colo.

He was Vice-President and Director of the Watson Oil Company, and President of the Mesa Oil Company; a Charter Member of the American Railway Engineering Association and a member of the Masonic Order, the Denver Country Club, and the Broadmoor Golf Club.

Captain Cooley was married at Toledo, Ohio, on July 7, 1887, to Emma E. Erwin, who, together with his sisters, Dr. Helen Cooley Palmer and Elizabeth Cooley, of San Marino, Calif., and a brother, Dr. Justus H. Cooley, of Plainfield, N. J., survives him.

Captain Cooley was elected an Associate Member of the American Society of Civil Engineers on March 7, 1894, and a Member on December 3, 1902.

EDWIN DURYEA, Jr., M. Am. Soc. C. E.[1]

DIED MARCH 31, 1930

Edwin Duryea, Jr., was born at Craigville, N. Y., on July 12, 1862. He was the son of Edwin and Hannah (Rumsey) Duryea. His family were Huguenots from Burgundy, France, who came to the United States in 1675 and settled in Long Island. Their descendants fought in the French and Indian Wars and, later, in the American Revolution.

Mr. Duryea spent his boyhood in New York State, attending the public schools and Cornell University, at Ithaca, from which institution he was graduated in 1883 with the degree of Bachelor in Civil Engineering. In

---

[1] Memoir prepared by Charles D. Marx, Past-President, Am. Soc. C. E., and Charles B. Wing and H. L. Haehl, Members, Am. Soc. C. E.

1890, he received the degree of Civil Engineer from the same institution. His first engineering work was on the Northern Pacific Railroad Bridge over St. Louis Bay at Duluth, Minn., under the late E. H. Beckler, M. Am. Soc. C. E., Assistant Engineer, and the late Alfred Noble, Past-President, Am. Soc. C. E., General Assistant Engineer in charge of bridges.

From March, 1886, to April, 1889, Mr. Duryea was employed under the late George S. Morison, Past-President, Am. Soc. C. E., as Assistant Engineer on the Rulo Bridge at Rulo, Nebr. (with the late B. L. Crosby, M. Am. Soc. C. E., as Resident Engineer), and as Assistant Engineer in charge of borings, surveys, etc., for the location of the Memphis Bridge over the Mississippi River, at Memphis, Tenn.; he also acted as Assistant Engineer in charge of river protection and reclamation for the Nebraska City Bridge over the Missouri River at Nebraska City, Nebr., and as Resident Engineer on the new Omaha Bridge (Union Pacific Railway) over the Missouri River at Omaha, Nebr. He was also employed under the late Mr. Noble, as Resident Engineer on the Cairo Bridge (Illinois Central Railroad) over the Ohio River, at Cairo, Ill.

During 1889 and 1890, Mr. Duryea was engaged on the location of the Hutchinson, Oklahoma and Gulf Railroad, in Kansas; on the Chicago and Western Michigan Railroad, and the Detroit, Lansing and Northern Railroad, Grand Rapids, Mich., and as Assistant Engineer on the construction of the Traverse City Extension of the Chicago and Western Michigan Railway.

From January, 1891, to May, 1893, he served as Contractor's Engineer with Washburn, Shaler, and Washburn, Civil Engineers and Contractors, in charge of topographic surveys at the lower end of Lake Michigan, harbor studies, etc., as well as on studies and plans for stockyards, packing houses, railroad layout, and water supply and sewerage systems near Chicago, Ill. Under the same employment he was also engaged on computations for bids for the construction of the New Croton Dam, New Croton Aqueduct, for the New York City water supply, and on the construction of the Third Avenue Cable Railway, in New York City, as well as in serving as Contractor's Engineer and Assistant Superintendent on a part of the New Croton Aqueduct. From May, 1893, to September, 1895, he acted as Superintendent for the same firm of contractors on the construction of the dams of the New Croton Aqueduct.

Beginning in September, 1895, Mr. Duryea was engaged on the design and construction of the New East River (Williamsburgh) Bridge, New York City, including investigations for live load, grades, tower foundations, etc., and the design of anchorages and similar structures. He was later Resident Engineer in charge of the Brooklyn half of the New East River Bridge until 1900, after which time he was engaged in private practice in New York City.

In 1902, he went to San Francisco, Calif., as Chief Engineer of the Bay Cities Water Company, and its allied interests, being engaged on investigation and design for city water supply and power development in connection with that Company until 1910.

Beginning in 1907, Mr. Duryea became associated (as Senior member) with the engineering firm of Duryea, Haehl, and Gilman, of San Francisco. He continued in that connection until 1920. His work during this period involved the construction of the system of the South San Joaquin Irrigation District in California, the completion of dams and power development on the Rio Conchos in Mexico, and many other important engineering works in connection with water supply, irrigation, and power development in California. During this time he was a member of the "Committee of Forty" appointed by the Mayor of San Francisco to advise on the rehabilitation of the city after the great fire of 1906.

Owing to ill health, Mr. Duryea retired from active practice in 1920 and remained thereafter at his home in Palo Alto, Calif.

He was always an active and enthusiastic member of the Society and participated whole-heartedly in its activities. He took his profession very seriously, was especially jealous of its honor, and punctiliously observant of its ethics. He was one of the Charter Members of the San Francisco Section of the Society, having served as its President during 1906. He was also a member of the Western Society of Engineers, of Chicago, and an Associate Member of the Institution of Civil Engineers, of London, England.

Mr. Duryea was a man of unusual force and personality, whose gentlemanly conduct and consideration of others impelled the respect and admiration of all those who knew him. The high order of his engineering work, the great interest which he took in his fellow-engineers, and the unselfish assistance which he rendered, particularly to young engineers with whom he came in contact, won for him the unreserved loyalty and friendship of all in his profession.

In December, 1888, he was married to Roberta Taylor, in Ithaca, who, with five children, Robert, Margaret, Anne, Philip, and Helena, as well as a sister, Mrs. Julia Tuthill, and a brother, Mr. Howard Duryea, survives him.

Mr. Duryea was elected a Junior of the American Society of Civil Engineers on February 2, 1887; an Associate Member on September 4, 1895; and a Member on February 2, 1898. He served as a Director of the Society from 1916 to 1918.

HARRISON ROBERT FEHR, Sr., M. Am. Soc. C. E.[1]

DIED MAY 30, 1930

Harrison Robert Fehr was born in Nazareth, Pa., on March 20, 1863, the son of Michael M. and Anna Caroline (Edmonds) Fehr. He was educated in the public schools of Nazareth and at the Moravian Parochial School.

From 1880 to 1883, Mr. Fehr was a teacher in the public schools. He then entered the Civil Engineering Department of the Lehigh Valley Railroad Company, acting, successively, as Rodman, Transitman, and Assistant Engineer, from 1883 to 1889. At the end of this period he was engaged in the private practice of engineering for a time and, in 1890, was again associated with the Lehigh Valley Railroad Company as Bridge Engineer.

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

The following year, 1891, he was elected City Engineer of Easton, Pa., and during his tenure of this office more than $250 000 was spent by this Department, under his supervision, on various municipal improvements. He served the city ably and well, furthering street paving, the building of macadam roads and retaining walls, the opening and grading of streets, and other improvements of a like nature. During this period his duties also included engineering for the Easton Board of Trade, the South Side Land Company of Easton, and for the Borough of South Easton.

In 1899, Mr. Fehr was appointed Engineer to the Easton Transit Company, and, except for short intervals, he was allied with this Company until 1920. His several positions during this period were as follows: 1901-1902, Chief Engineer, Trenton and New Brunswick Electric Railway; 1902-1904, Contracting Engineer; 1904-1913, President and General Manager, Easton Transit Company; and 1913-1920, President and General Manager, Lehigh Valley Transit Company, and Vice-President, The Pennsylvania Light, Power, and Gas Company. Mr. Fehr was also a promoter of the construction of the William Penn Highway between Bethlehem and Allentown, Pa.

In 1920, he again entered private practice as a Consulting Engineer in which he continued until his death. He was also active in the wholesale coal and coke business.

Mr. Fehr was a member of the Pennsylvania Street Railway Association (President, 1912), Lehigh Valley Engineers Club, the Pennsylvania Society, the Sons of Veterans, the Pennsylvania Society of Sons of the Revolution, the National Geographic Society, the Masonic Orders, and the Benevolent and Protective Order of Elks. He was also a member of the Livingston Club, the Country Club of Northampton County, and an Honorary Member of the Writers Club of Allentown. He was a Trustee of St. Luke's Hospital, Bethlehem, Pa., and a Director of the Penn Securities Company. His political interests were Republican. He was a life-long member of the Presbyterian Church.

He was married on August 6, 1881, in Nazareth, to Achshaw Walter who, with one son, Harrison Robert, Jr., survives him.

Mr. Fehr was elected a Member of the American Society of Civil Engineers on October 5, 1898.

SAMUEL MORSE FELTON, M. Am. Soc. C. E.[1]

Died March 11, 1930

Samuel Morse Felton was born in Philadelphia, Pa., on February 3, 1853. He came of old English stock, his ancestors having settled in Salem, Mass., in 1633. His earliest paternal ancestor was Nathaniel Felton, Ensign and Lieutenant in the Salem Foot Company, who came to America from Great Yarmouth, Norfolk, England. His father, Samuel Morse Felton, was Superintendent of the Fitchburg Railroad in 1843, and, in 1861, as President of the Philadelphia, Wilmington, and Baltimore Railroad Company, arranged for the secret passage of Abraham Lincoln from Harrisburg, Pa., to Washington, D. C., at the time of his first inauguration. He afterward became President of the Pennsylvania Steel Company. His mother, Maria Low (Lippitt) Felton, was the seventh in descent of Roger Williams, Captain in King Philip's War, and Governor of Rhode Island Colony, in 1654.

---

[1] Memoir prepared by W. G. Lerch, Vice-Pres., C. G. W. R. R., Chicago, Ill.

Mr. Felton's inclination and early training were along railroad lines. He was educated in private schools in Philadelphia, and, for two years, took an engineering course at the Pennsylvania Military Academy, at Chester, Pa., working during vacations as Rodman on the Chester Creek Railroad. After leaving the Academy at Chester he spent one year in the field with a locating party on the Lancaster Railroad, in New England, as Leveler and Assistant Engineer. Realizing the value of further technical training he entered the Massachusetts Institute of Technology, at Boston, Mass., and was graduated in 1873 with the degree of Civil Engineer. After his graduation, because of his earlier experience in the field, he was able to accept a position as Chief Engineer of the Chester and Delaware Railroad, a small line being constructed by the Reading Railroad Company along the Delaware River.

His ambition was to become the operating head of a railroad system, and, in 1874, at the age of twenty-one, he was appointed General Superintendent of the Pittsburgh, Cincinnati, and St. Louis Railroad Company, with which he remained until 1882. In 1881, Mr. Felton was also made General Superintendent of the Little Miami, and Cincinnati and Muskingum Valley Railroad Companies. In 1882, he became General Manager of the New York and New England Railroad Company; in 1884, Assistant to the President of the New York, Lake Erie, and Western Railroad Company, and General Manager of the New York, Pennsylvania, and Ohio Railroad Company, both of which are now a part of the Erie System. In 1885, he was made Vice-President in Charge of Traffic, later taking over, in addition, the duties of Vice-President in Charge of Operation. He remained in these positions until 1890, when he was elected President of the East Tennessee, Virginia, and Georgia Railway Company and, later, President of the Alabama Great Southern Railroad and the Cincinnati, New Orleans, and Texas Pacific Railway Companies. In 1893, when the latter company went into a receivership, Mr. Felton was appointed Receiver of that property by former President William H. Taft, who, at that time, was on the Federal Bench in Cincinnati, Ohio. He also in that year became Receiver of the Columbus, Sandusky, and Hocking Railway Company and the Kentucky and Indiana Bridge Company. He directed the operations of these properties until 1899, when the late E. H. Harriman secured control of the Chicago and Alton Railroad and, finding complete rehabilitation necessary, decided that Mr. Felton was best fitted to undertake the work. He prevailed upon him to come to Chicago, Ill., as President of the road, where he remained until 1907. During this time he rebuilt much of the line, installed modern facilities and equipment, and the Alton Railroad, when he left it, was as well equipped as any road in its territory. In December, 1907, he was elected President of the Mexican Central Railway Company and the Mexican American Steamship Company, continuing in those positions until the Mexican Government merged all the principal railroads in the Republic into the National Railways of Mexico.

In 1909, Mr. Felton returned to the United States and became Chairman of the Tennessee Central Railroad Company. In the fall of that year after making an examination and report on the road for the re-organization managers he was elected President of the Chicago Great Western Railroad Company and its affiliated lines. He held this office until the end of 1925, when he was made Chairman of the Board of Directors, in which position he remained until his death. From 1912 to 1914 he was also Co-Receiver and President of the Pere Marquette Railway Company. When Mr. Felton took charge of the Chicago Great Western Railroad Company, it had just emerged from a receivership, but within a period of six years he completely rehabilitated the road and its equipment and placed the property on a dividend-paying basis. During the period of Federal operation of railroads incident to the World War, the road was not maintained, and when it was turned back to its owners in 1920, it was in a bad state of repair with its traffic diverted to other roads, necessitating doing over again what Mr. Felton had begun in 1910—the reconstruction and rehabilitation of the road and its equipment and the task of getting back its traffic. He went vigorously ahead and had practically completed the work with prospects for renewed prosperity of the Company in sight when illness assailed him, and he was compelled to relinquish the active direction of the railroad to others.

Mr. Felton maintained his interest in railroad problems, and was a member of the Executive Committee of the Association of Railway Executives, of the Western Association of Railway Executives, and of the Western Railway Committee on Public Relations. He was also a Director of the American Railway Association and President of the Western Railroad Association. During his railroad career he served on several important engineering commissions, having acted as Chairman of the Engineering Commission of the City of Cincinnati to select the location for a new water-works, and, in 1900, he was engaged by the Commercial Club of St. Louis, Mo., to report on a municipal bridge project. At different times, he made reports for banking interests and re-organization managers on 23 railroads aggregating more than 32 000 miles.

Mr. Felton's most spectacular service was probably in connection with the Spanish-American and the World Wars. During the former, while in charge of the Cincinnati, New Orleans, and Texas Pacific Railway, his road moved a larger number of troops than any other single line. At the outbreak of the war it handled the first troop movement south to Chattanooga, Tenn., which consisted of 31 trains leaving Cincinnati 10 min. apart and reaching Chattanooga on time, without delay or accident; and this was accomplished over a single-track road.

In June, 1916, while acting as President of the Chicago Great Western Railroad, he was requested by the War Department to organize troops to operate the railroads in Mexico in the event the United States Army found it necessary to advance south of the border. His title was Consulting Engineer and Adviser to the Chief of Engineers, United States Army. He went zealously to work and, in a short time, engaged a sufficient number of railroad men, familiar with Mexican railways, to fill the most important positions thereon, and arranged under option, if needed, for locomotives, construction and wrecking trains, rails, and cross-ties to replace destroyed track, telephone and telegraph equipment, and other material. In June of that year he joined the Staff of General Funston, at San Antonio, Tex., and reported sufficient men and material ready for the initial movement of troops. This, however, became unnecessary as the tension between the two countries relaxed.

In the meantime the question of forming a reserve force of railroad men in the Engineer Corps was considered by the War Department, and Mr. Felton was requested to plan its organization. While this work was progressing, the United States entered the World War and the original plan was enlarged upon, as it was found necessary to organize, at once, nine regiments. The work of equipping and drilling these regiments was hurried, and they were among the first American troops to land in France. Mr. Felton was appointed Director General of Railways, United States Army, the title later being changed to Director General of Military Railways. He took up his residence in Washington and went actively to work organizing additional regiments, being empowered by the Secretary of War to make contracts and purchases, at his own discretion, without going through the customary formalities. Two days after his appointment as Director General of Railways, he contracted for the building of three hundred locomotives and in twenty working days thereafter the first one was crated ready for shipment. The total amount of contracts made under his direction was $612 000 000, covering: 3 750 standard gauge locomotives; 1 547 narrow-gauge locomotives; 91 519 standard-gauge cars; 8 930 narrow-gauge cars; 823 locomotive cranes; 76 gantry cranes; and materials for more than 100 water stations, 749 345 tons of rail (the equivalent of 5 951 miles of single track), and other items of like proportion. By careful buying a saving of more than $33 000 000 was made in connection with these contracts.

In addition to the purchasing of materials and supplies, 84 000 transportation men were organized in battalions and regiments for service overseas and, at the time of the Armistice, 70 000 transportation men were in France and 14 000 more ready to sail.

In 1918 Mr. Felton made a trip of inspection to France, traveling among all the battle lines. On May 23 of that year, he was made Vice-President of the Port and Harbor Facilities Commission of the United States Shipping Board, and, in 1918, became Acting Chairman, performing the duties of the office until he left Washington. He resigned as General Director of Military Railways on December 1, 1918.

Mr. Felton emerged from the war without military rank. He was twice offered a commission as Brigadier General, but refused, believing that a place in the military organization would prove a hindrance rather than a help in his work. He was awarded the Distinguished Service Medal of the United States Army, the first one granted to a civilian. In further recognition of his war work, the French Government made him an Officer of the Legion of Honor.

After the close of the war he was appointed Honorary Adviser to the Army Industrial War College, a member of the War Department Business Council, and Chairman of the Committee on Military Affairs of the Association of Railway Executives.

Notwithstanding the heavy responsibilities imposed upon him by his railroad duties, Mr. Felton found time to interest himself in many outside mercantile and financial activities, the success of which bore witness to his ability, and added luster to the corporations with which he was identified. He was a Director of the Central Trust Company of Illinois, the Central Securities Company, the Peoples Trust and Savings Bank of Chicago, the Equitable Life Assurance Society, the Chicago Surface Lines, and the Colonial Land Company. He was a member of the Western Society of Engineers, American Railway Engineering Association, Franklin Institute, American Legion, Civil Legion, Society of American Military Engineers, Veterans of Foreign Wars, Ohio Society, Sons of the Revolution, Ohio Society of Colonial Wars, and the New England Society, in New York; the Commercial Club of Chicago, Chicago Club, Chicago Athletic Association, Saddle and Cycle, and Old Elm Golf Club, of Chicago; the University Club of New York and the Commercial Club of Cincinnati.

There was conferred upon him the degree of Doctor of Laws by the Pennsylvania Military Academy and by Marietta College, of Marietta, Ohio. He was a member of the Corporation of the Massachusetts Institute of Technology.

Mr. Felton was married in 1880, to Dorathea Hamilton, of Pittsburgh, Pa. Mrs. Felton died in 1923. They had three daughters and one son.

Mr. Felton was elected a Member of the American Society of Civil Engineers on January 4, 1882.

ROGER MORSE FREEMAN, M. Am. Soc. C. E.[1]

Died January 21, 1925

Roger Morse Freeman, the son of John R. Freeman, Past-President and Hon. M. Am. Soc. C. E., and Elizabeth Farwell (Clark) Freeman, was born in Winchester, Mass., on July 20, 1892. His ancestry on both sides was English Colonial, dating back to Governor Winthrop's Boston Colony of 1636.

When he was about four years of age his father moved with his family to Providence, R. I., where the boy attended public and private schools, finishing his common school education as a graduate of Hope High School in 1908. During his high school days he was Manager of both baseball and football teams, playing on the eleven that won the year's New England High School championship in football. He carried this activity with him into college athletics where it is said some members of the faculty thought that for a time it overshadowed his interest in textbooks and formulas. However, Mr. Freeman claimed in after life, that this time was well spent because of the appreciation it gave him of teamwork and human nature. Also, this interest in athletics from earliest boyhood developed an uncommonly fine physique which, in later years, could endure any required amount of overload.

---

[1] Memoir prepared at the Headquarters of the Society from information supplied by John R. Freeman, Past-President and Hon. M. Am. Soc. C. E.

Mr. Freeman entered the Massachusetts Institute of Technology in 1909, and was graduated with the class of 1913, but took his degree in Electrical Engineering with the class of 1914, meanwhile having spent a term at the Technical University of Charlottenburg, Germany, where he majored in the theory of internal combustion engines. Prior to this, he had spent a long summer vacation in Germany, in order to gain greater familiarity with the language than the college course permitted. The breaking out of the World War in 1914 prevented carrying out his intentions of further study in both Germany and France.

His practical experience in engineering began at the early age of fifteen, when being large and strong for his age, he obtained a summer job as Rodman in a surveyor's party on the New York Board of Water Supply. One of his college vacations was spent as Assistant in an engineering party in the eastern foothills and slopes of the Sierra Nevada Mountains in California, on the construction of the Los Angeles Aqueduct. Another was spent as a Machinist in the Marine Repair Shops of the Atlantic Works in East Boston, Mass.

After graduation from college Mr. Freeman spent about a year in his father's engineering office on water power development studies and designs. He then concluded that he could build more securely for the future by going where he would be forced to depend purely on his own efforts, or, as he expressed it, he desired to "try making progress on his own legs for ten years, after which, he would be glad to join forces with his father." In following out this idea, being always ready to enter an open door, without waiting for something precisely in his line of Electrical Engineering, he first took a Time-Keeper's job with the Turner Construction Company on the construction of a large factory building at Thomaston, Conn., in the belief, which later he often expressed, that this was the best possible post-graduate course for a young engineer, since it took him to every part of the work at least twice a day; gave him an opportunity to see how much could be done in a day; made him familiar with obstacles; and, if he was sympathetic, taught him the grievances of foremen and workmen and their psychology and reactions.

He next became Construction Engineer for the Chase Rolling Mills and Waterbury Manufacturing Company of Waterbury, Conn. At the time, this Company was concerned in the rush of war work, employing 10 000 workmen. Mr. Freeman designed and supervised the construction of several large multistoried reinforced concrete factory buildings, and so gained the confidence of his employers that when later they decided to build a large machine shop, to be used in fabricating their own heavy special machinery for working brass and copper tubes, plates and similar articles, he was chosen to supervise the design and construction. He was given but one specification. As the President of the Company put it: "We need a machine shop about 400 by 100 ft. I am too busy with war problems to discuss details. Go to it! And make it the best of its size in New England".

As this undertaking neared completion, Mr. Freeman was summoned into conference with the Secretary of the Navy, the United States then having entered the World War. As a result he was requested to visit certain large steel works under construction for the manufacture of certain forgings most urgently needed, and to report, as one familiar with rush work, what, in his judgment, was causing the very apparent delay. On his return he reported: "Making decisions in committee meetings instead of 'off the bat'" * * * "Too many Bosses." * * * "Perhaps too much gold braid." A few hours later he was recalled and asked whether he would take these responsibilities, being given sole and full authority. In six months, employing three shifts, through one of the worst winters on record, he had completely built the plant, ready to forge crank-shafts and guns.

Meanwhile, as a result of his success in getting all hands to pull hard together, he was dispatched here and there to help speed up other constructions for war material that lagged. Sketches for starting rush work on a tall heat-treatment tower were made on the back of an envelope, and the structure was completed without stopping the progress of work in the old tower beneath it.

After the Armistice, the Navy officials had strong beliefs about the need for a larger Navy, which, in part, found expression in the building of an armor-plate plant at Charleston, W. Va. This plant was to serve for research in a large way and, at the same time, to check the actual costs on bids submitted by the great steel works, which some members of Congress had felt were unduly high.

Inasmuch as the technology of steel-making and steel-plant design were somewhat outside the regular curriculum of the Navy Department, and as Mr. Freeman had shown such resourcefulness and success at war emergency plants, he was again given a free hand. In two years, starting with practically an open lot and blank drafting paper, and aided by the engineering organization which he had built up, beginning at Waterbury, and also by the friendly co-operation from many sources within the Navy Department, he was able to complete a plant for armor-plate and heavy forgings, perhaps without a superior of its size in the world. This had successfully poured its first heats a short time prior to the first Disarmament Conference, since which time it has stood idle. He presented a paper[2] descriptive of this plant before the American Society of Mechanical Engineers in December, 1920.

Following his natural bent, Mr. Freeman had hoped to continue in metallurgical structural lines, but as no openings were found in the depressed condition of this industry after the war, he opened an office in New York, N. Y., and turned again to hydro-electric construction, in which he had had previous experience in his father's office. In this connection, his ability and experience, his friendliness with builders of hydraulic and electrical machinery, and his unbounded patience in attention to details, quickly brought to successful completion a plant of about 12 000 h.p., for the Insull interests, on the Tippecanoe River, at Norway, near Monticello, Ind., a model of low cost and efficiency.

The end found him driving ahead on the design and construction of a second hydro-electric plant of about 15 000 h.p. for the same interests near Oakdale,

---

[2] "The Armor-Plate and Gun-Forging Plant of the U. S. Navy Department at South Charleston, W. Va.", *Transactions*, Am. Soc. Mech. Engrs., Vol. 42 (1920), p. 983.

Ind., on the Tippecanoe River, with more than 500 men employed on a difficult piece of rush work under winter construction. The confidence of his employers may be judged from the fact that the formal contract for his services in designing and supervising the Oakdale plant had not been signed to the time of his death, although a draft had been prepared. He was proceeding with design and supervision of construction to the satisfaction of all concerned under his oral statement that he could produce plans as fast as the construction forces could be ready for them. The reservoir created by these works was subsequently named Freeman Lake in his honor.

In ten years, Mr. Freeman under exceptional conditions brought about by the World War had accomplished more in engineering design and construction than most men find an opportunity to achieve in a life time. The cost of structures completed under his design and supervision, during the brief practice of his profession, aggregated a total of about $25 000 000. Plans under way for joining forces with his father in consulting work on larger enterprises and for taking construction contracts, were cut short by his untimely death.

While he was at Thomaston, Mr. Freeman was married on March 2, 1918, to Mary Elizabeth Bradstreet, of that town, a lineal descendant of Colonial Governor Simon Bradstreet and of Anne Bradstreet, the eminent poetess of early Colonial days.

His death, when he was but 33 years of age, was caused by heart failure following an operation for appendicitis. Besides his widow and two children, Anne Bradstreet and Roger Morse, Jr., his father and mother, he is survived by four brothers: Hovey T. Freeman, Evert W. Freeman, Clarke Freeman, and John R. Freeman, Jr., and a sister, Mary Elizabeth Freeman, of Providence.

He was a member of the American Institute of Electrical Engineers, the American Society of Mechanical Engineers, the American Association of Engineers, and the Engineers' Club of New York City.

Mr. Freeman was elected a Member of the American Society of Civil Engineers on January 15, 1923.

HORACE DELPHOS GATES, M. Am. Soc. C. E.[1]

DIED JANUARY 8, 1931

Horace Delphos Gates was born at Boston, Mass., on February 12, 1847. He never tired of recalling his early fondness for mathematics and natural philosophy when relating how, in making a choice of a vocation, he was attracted to surveying and engineering. Through mutual friends he was brought into contact with Mr. L. H. Shortt, a railroad engineer, originally from Canada, but at that time (sometime in the Sixties) already well established on work for several lines of railroads with terminals at San Francisco and Oakland, Calif. Mr. Gates became an Assistant to Mr. Shortt with part time in the field and part time in the office and also with opportunity to continue his studies in mathematics and engineering under the personal supervision of his Chief.

---

[1] Memoir prepared by C. E. Grunsky, Past-President, Am. Soc. C. E.

It was during these years with Mr. Shortt that his surveys of railroads and proposed railroad extensions brought Mr. Gates into pleasant relations with the late E. A. Flint, M. Am. Soc. C. E., another railroad engineer of high attainments, who had spent eleven years in Chile where he has been employed on the construction of the railroads then being built by Mr. Harry Meiggs.

In 1867, Mr. Gates was called into the office of the City and County Surveyor of San Francisco by the then incumbent, Mr. William P. Humphreys. This promised to be a fairly permanent position, but when, a year later, Colonel Flint said "I want you", the prospective railroad work proved too alluring and Mr. Gates took charge of the location of about 20 miles of railroad from tide-water at Wilmington to Los Angeles, Calif.

Subsequently, he was again with Mr. Shortt on the location and building of the California Pacific Railroad from Vallejo to Sacramento, Calif. This was followed in 1870 by work on the Central Pacific Railroad under Chief Engineer S. S. Montague, in the northern part of California, and, later, under the late William Hood, M. Am. Soc. C. E., with constantly increasing responsibility.

This and other railroad construction kept Mr. Gates active in his favorite line of work until 1874 when he was called into the San Joaquin Valley, California, in charge of the extension of the San Joaquin and Kings River Canal from Los Baños to Orestimba Creek. This irrigation experience naturally led him into the newly organized State Engineer Department of California, under William Hammond Hall, M. Am. Soc. C. E., State Engineer. In 1878, Mr. Gates was Transitman under the late Isaac W. Smith, M. Am. Soc. C. E., on a topographic survey of Sacramento River from Knights Landing to the head of Suisun Bay. He was then assigned to hydrographic studies on the same river which were being made by the Engineer Corps of the United States Army, and then came other work with the Government engineers at various places on the Pacific Coast, covering a total period of three years.

A few months at a mine in Nevada and, again, some railroad surveys under Mr. Lyman Bridges for the Gould Syndicate in California, and then again five years on mining and railroad work in Nevada added variety to Mr. Gates' activities. Scarcely had he returned to San Francisco, when he was once more made an Assistant in the office of the City and County Surveyor, Mr. S. Harrison Smith, where he was retained for six years, or more.

Thereafter, he assisted in making the studies for San Francisco's new sewer system and, when the office of City Engineer was established, under a Board of Public Works in 1900, Mr. Gates was appointed as one of the Assistants to the writer who was then City Engineer. Mr. Gates retired from the City Engineer's Office in 1913, at which time he gave up active practice. He died at Sheridan, Calif., on January 8, 1931.

He is survived by his widow, *née* Grace Birdsall, to whom he had been married on April 29, 1902.

His was a charming personality. He was modest to a degree and every one who came in contact with him became his friend. In his professional work he was conscientious, energetic, and able, enjoying a long life of engineering activity followed by a few well-earned years of retirement.

Mr. Gates was elected a Member of the American Society of Civil Engineers on March 7, 1883.

THOMAS GOLDING GERDINE, M. Am. Soc. C. E.[1]

Died October 31, 1930

Thomas Golding Gerdine was born at West Point, Miss., on June 2, 1872, the son of Dr. John Gerdine and Susan (Golding) Gerdine. He had two brothers and four sisters, all of whom are living and engaged in professional or educational work. Dr. Gerdine moved his family to Athens, Ga., in 1876, and continued the practice of medicine and the education of his children at that center.

After the completion of his elementary and High School studies at Athens, Mr. Gerdine entered the Engineering Department of the University of Georgia and was graduated in 1891 with the degree of Bachelor of Engineering. During his college course he was active in athletics and in fraternity affairs, being a member of the Sigma Alpha Epsilon Fraternity.

Shortly after his graduation he entered the public service and all his activities from that time were either in municipal or Federal Governmental work. His first position was as Assistant to the City Engineer of Athens, and he was placed in charge of field surveys, particularly the location and grading of streets and surveys for, and the inspection of, city sewerage and water-works systems.

On July 1, 1893, Mr. Gerdine joined the United States Geological Survey as Recorder in a topographic field party, working on the Santa Monica, Calif., Quadrangle. On May 3, 1894, he was appointed Assistant Topographer in the United States Geological Survey and, with the exception of his World War service, continued his work in the civil service of the Federal Government until his death. On April 22, 1895, he was transferred to the General Land Office and served for two years as Examiner of Surveys, acting as Federal Representative in the examination and approval of land surveys under the old contract system. On July 12, 1897, he returned to the Geological Survey, served as Party Chief in the extreme western part of the United States and in Alaska, and worked on difficult topographic and geodetic survey projects.

From May 1, 1899, to January 17, 1908, he was assigned to the Alaskan Division of the Geological Survey. He accompanied the party making the first survey of Chandalar River in interior Alaska, crossing a portage into the

---

[1] Memoir prepared by C. H. Birdseye, M. Am. Soc. C. E.

Koyukuk and descending that stream to its mouth, thence going to Nome, where, while awaiting transportation to the States, he supervised the making of the first map of the then newly discovered goldfields. Mr. Gerdine also conducted extensive surveys in the Copper River Region, Seward Peninsula, and Cook Inlet Basins in Central Alaska. Throughout his entire Alaskan service he showed not only marked professional ability, but all those other qualities, physical and personal, that are demanded of a leader of parties engaged in field work under difficult and unusual pioneer conditions.

Several of his assignments were with geologic field parties, which gave him a good conception of geological formations so essential to a mountain topographer's work. His experience covered all phases of topographic and geodetic surveys and fitted him especially for his later administrative work. In fact, he acted on the assumption that he should know all technical details and be able to prosecute actively any field operation which was under his supervision. In this connection of course he was overworked, but his physical and mental energy, coupled with a strong physique, enabled him to "stand the gaff" until shortly before his death. Major Gerdine falls more nearly in the category of men who "worked themselves to death" than any other man known to the writer.

On July 23, 1908, he was appointed Geographer in charge of the Pacific Division of the Geological Survey with complete administrative and technical charge of topographic and geodetic work in the Pacific Coast States and Hawaii. With the exception of his war service and with a few minor changes in areas, he continued in this administrative position until his death. In this work he came in contact with State and Federal officials, earned their admiration and respect, and gained the reputation of being the embodiment of cooperation.

He was commissioned a Major in the Engineer Officers' Reserve Corps on September 16, 1917, and was assigned to administration and supervision of military mapping, first in New Mexico and Texas, and, later, along the Atlantic Coast in Virginia and at some of the Eastern artillery training camps. He was discharged from the Army on March 31, 1919, and returned to his former administrative position on the Pacific Coast, making his home at Sacramento, Calif. At Sacramento, he was interested in the work of the Local Section of the Society and was always active in engineering and civic affairs. In his official work he brought to a successful conclusion the extremely difficult detailed mapping of the entire Hawaiian Islands.

Major Gerdine was twice married. His first wife was Frances E. Bishop, of Washington, D. C., to whom he was married in 1907 and who died in 1920. His second wife, to whom he was married in 1923, was Marguerite N. Rowell, of Falls Church, Va. He is survived by Mrs. Gerdine and three children, Suzanne, Eleanor, and Thomas F. Gerdine.

Major Gerdine was elected a Member of the American Society of Civil Engineers on June 6, 1921. He was also a member of the Society of American Military Engineers.

THOMAS NIXON GILMORE, M. Am. Soc. C. E.[1]

Died January 17, 1930

Thomas Nixon Gilmore was born in Manhattan, Ind., on December 10, 1863.

In 1880, Mr. Gilmore entered the service of the Vandalia Railroad Company, a part of the Southwest System of the Pennsylvania Railroad Company, as Special Apprentice in the Mechanical Department. He continued with this railroad until 1902, being employed, successively, as Draftsman, Engineer of Motive Power, Round-House Foreman, Division Foreman, and also, on various assignments, including studies in connection with fuel, steam heat, and oil.

In 1902, he entered the service of the Terminal Railroad Association of St. Louis, Mo., as Mechanical Superintendent in charge of the Mechanical and Car Departments. This position carried with it responsible charge of the design, construction, and equipment of extensive mechanical improvements preparatory to the World's Fair which was held in St. Louis in 1904. These improvements included locomotive shops and roundhouses, coaling plants, power houses, etc.

Mr. Gilmore was next engaged as Assistant General Superintendent of Motor Power for the Chicago, Rock Island and Pacific Railway Company during 1904 and 1905. In this position a large part of his time was devoted to improvements in standards of motor power facilities, in addition to the regular routine work connected with operation.

In 1905, he entered the service of Westinghouse, Church, Kerr and Company, as Engineer in charge of specific undertakings. The scope of this organization is well known and covers a great variety of engineering and construction operations, including railway lines, power facilities, industrial plants, and public utilities. The work under Mr. Gilmore's direction covered a large number of railway shops and terminals, including the West Springfield Shops of the Boston and Albany Railroad Company; the improvement and electrification of the Depew Shops of the New York Central and Hudson River Railroad Company; locomotive shops and power-house and incidental work at Silver Grove, Ky., and at other points, for the Chesapeake and Ohio Railroad Company; shops, power-house, and water supply at Havelock, Nebr., and West Burlington, Iowa, for the Chicago, Burlington and Quincy Railroad Company; the Ogden Shops of the Canadian Pacific Railway Company, near Calgary, Alta., Canada; and the Communipaw engine terminal at Jersey City, N. J., for the Central Railroad Company of New Jersey. His other activities included inspections and reports on the improvement of shops and terminal facilities for a number of trunk-line railroads.

Subsequently, Mr. Gilmore became Managing Engineer, giving both detailed and general supervision to works similar to those outlined. Later, he became Vice-President and Chief Engineer, and also a Director, of Westinghouse, Church, Kerr and Company, Incorporated. The activities of this

---

[1] Memoir prepared by James C. Boyd and Edwin J. Beugler, Members, Am. Soc. C. E.

Company were accented considerably during the period of the World War, and Mr. Gilmore's position as Chief Engineer contributed toward the expansion of the engineering and construction organization, which covered a large amount of industrial plant construction, warehouses, and special work, much of which was done directly for the United States and Canadian Governments in co-operation with the Army officers. A citation given to the Company after the war by the United States Government was due in a large degree to the efficiency of the organization developed under Mr. Gilmore as Chief Engineer.

In 1920, Westinghouse, Church, Kerr and Company was merged with Dwight P. Robinson and Company, continuing along the same lines of endeavor, and Mr. Gilmore became Consulting Engineer in the combined organization, giving particular attention to railroad shops, locomotive terminals, and power stations. In 1921, he entered private practice as Consulting Engineer, specializing on the lines in which he had been most actively engaged and thus continued until the time of his death.

Mr. Gilmore's contact with a great number of railway systems in the United States and Canada and his habits of observation and analysis fitted him for rendering unusual service in the development of designs and construction of railway facilities for any given location. Moreover, his faculty of co-operation with the various departments of the railroads for which he was doing work, produced results which reflected the best ideas of all concerned.

He was a member of the American Society of Mechanical Engineers, and the Structural Engineers' Association of Illinois. His connection with social organizations included the Engineers' Club of New York City, and the Echo Lake Golf Club of New Jersey.

Mr. Gilmore was elected a Member of the American Society of Civil Engineers on April 7, 1915.

BENJAMIN MORTIMER HALL, M. Am. Soc. C. E.[1]

Died November 19, 1929

Benjamin Mortimer Hall was born near Wynnsboro, in Fairfield County, South Carolina, on January 31, 1853, the son of Dr. Nathaniel Hall and Nancy (Boulware) Hall. The family moved to Georgia in the boy's childhood and located in Webster County, where his early youth was mainly spent. He attended the University of Georgia and was graduated in 1876 as Bachelor of Engineering. Following his graduation, he became Professor of Mathematics in the North Georgia Agricultural College at Dahlonega, in the Blue Ridge Mountain section of the State.

During the four years of his incumbency there Professor Hall became interested in the surrounding mining and hydraulic operations which influenced his studies in securing the Post-Graduate Degrees of Civil and Mining Engineer bestowed by the University of Georgia in 1885 and which gave bent to his later professional activity and connection with mining work. Thus, from 1880 to 1890, he served as Mining Engineer and Superintendent of gold mines and marble quarries of Northeast and Northwest Georgia and established a reputation for thoroughness, initiative, character, and professional engineering ability of the highest order.

---

[1] Memoir prepared by a Committee of the Georgia Section consisting of W. C. Spiker, *Chairman*, J. Houstoun Johnston, C. M. Strahan, and S. B. Slack, Members, Am. Soc. C. E.

In 1890, together with his brothers, Max R. Hall, M. Am. Soc. C. E., and Mr. James R. Hall, he organized the firm of Hall Brothers, Engineers, with offices in Atlanta, Ga.; and through this firm, as its head and senior member, he served most acceptably a growing clientage in Georgia, Florida, Alabama, the Carolinas, Tennessee, Texas, New Mexico, and, later, in Porto Rico.

As Consulting Hydrographic Engineer for the United States Geological Survey (1896-1903), Mr. Hall directed the organization and active field operations for stream flow, run-off, and other significant water-power data on the principal streams and water-sheds of Georgia, Alabama, the Carolinas, and Tennessee. Under joint authorship with his brother, Mr. Max R. Hall, two important volumes, "The Water Powers of Georgia," and "The Water Powers of Alabama," published by the U. S. Geological Survey and by the respective States, record and attest the sound professional judgment and clarity of presentation with which Mr. Hall handled this large mass of basic and important hydrographic and water power information. To these larger publications, he added a goodly array of special reports and a store of personal records and memoranda which deservedly ranked him as an outstanding authority in hydrographic matters among the engineers of the Southeastern States.

Mr. Hall served from 1904 to 1907 as Supervising Engineer for the United States Reclamation Service. Connected therewith he built in the Southwest, the Hondo, Carlsbad, and Leesburg Projects, and negotiated the terms of the Mexico-Rio Grande Treaty at El Paso, Tex. He prepared the original plans for the Elephant Butte Dam and Rio Grande Project and supervised the settlement of all water-right disputes arising from this large storage and irrigation enterprise. The lake formed by Elephant Butte Dam which was named Lake B. M. Hall in his honor, is the largest artificial reservoir in the United States.

From his Texas and New Mexico experience, Mr. Hall was called as Chief Engineer to study, supervise, and construct (1908-1910) the widespread irrigation development for the Porto Rican Irrigation Service. This work has received distinct professional approval for its comprehensive planning, durability, and efficiency.

Returning to Atlanta in 1911 he acted for two years as General Manager for the Amicalola Marble Company in Pickens County. He also became active at this time in a growing consulting practice related to water power development and flood-water damage problems. The City of West Point, Ga., profited by his sound advice and by his proposals for flood protection.

Subsequent to 1917, Mr. Hall's services as Consulting Engineer were in great demand. He served on important boards of arbitration involving engineering issues, and was sought as an expert witness in many controversies. Among these should be noted long hearings in Raleigh and Durham, N. C., and in New York, N. Y., and Washington, D. C.

Mr. Hall was a member and supporter of the American Institute of Mining and Metallurgical Engineers. He was Chairman of the Water Power Committee at a session of the National Conservation Congress and performed important committee services for many engineering and civic organizations.

He was also a member of the Atlanta Board of Consulting Engineers for a number of years. His professional attainments were recognized by his charter membership in the Georgia Academy of Science, representing therein the Engineering Profession. He was always liberal of his time and means in civic and other altruistic movements. His support carried the weight of a strong clean-cut and high-minded personality which attracted friends to himself and the causes he espoused. He was an active member of the Civitan Club and other social organizations which he greatly enjoyed.

Mr. Hall had a systematic practice which can be commended to all his professional colleagues. He seldom failed to enter each day in a special pocket notebook brief data of the day's happenings; where he was, names and addresses of persons whom he met, and subjects of business matters discussed. Thus, his office library contained a special shelf of notebooks duly labeled which made a well-nigh complete inventory of his business contacts and matters handled. It served to solve many important problems regarding new projects or additions to old projects the original details of which had grown dim in memory through lapse of time. These notes furnished an explanation of the systematic thoroughness with which he executed all professional matters committed to his care, and show why he was a most convincing and accurate witness before Court, jury, or conference board.

Mr. Hall always showed unswerving loyalty and active interest in the work of the Society. He exemplified by precept and example its professional ideals and was a factor in advancing the standing of the Engineering Profession in public opinion. He was active in the organization of the Georgia Section of the Society and a constant supporter of its efforts. He took much interest in the young engineers of the Student Chapters at the University of Georgia and the Georgia School of Technology and honored them with timely addresses and advice.

He was long a devoted member of the Methodist Episcopal Church and a Steward and Trustee of the St. Mark's Church at the time of his death.

Mr. Hall was married on January 5, 1881, to Kate Chamberlin, of Weston, Ga., who died in August, 1929. They had two sons and a daughter, Mrs. Brainard K. Clapp. His sons, Benjamin Mortimer, Jr., and Warren Easterly, Assoc. Members, Am. Soc. C. E., in 1917 joined with him professionally under the firm name of B. M. Hall and Sons. He remained the active head of the firm until his death.

A kindly gentleman who attracted and enjoyed friends, he was genial, sagacious, clear-minded, professionally able, energetic, thorough, sympathetic, conscientious, and fair; such is the record which he has left.

Mr. Hall was elected a Member of the American Society of Civil Engineers on February 6, 1901.

ALFRED ERNEST HARVEY, M. Am. Soc. C. E.[1]

DIED JULY 3, 1930

Alfred Ernest Harvey, the son of Alfred and Elizabeth George Harvey, was born on September 16, 1869, at Waverly, Ill. He was the eldest of four brothers. His father, who was a teacher in the public schools, moved the family, while the children were still quite small, to Paris, Ill., which was always considered by Mr. Harvey to be his home.

He received the degree of Bachelor of Science from the University of Illinois in 1891, and that of Civil Engineer from the same institution in 1894. His first work after leaving college was on surveys with the firm of Cole, Alvord, and Shields, of Chicago, Ill. Later, he was employed in the office of the Chief Engineer of the Chicago, Burlington, and Quincy Railroad Company as Draftsman, and, still later, was appointed City Engineer of Urbana, Ill. In 1892, he was Assistant Engineer of Sewers and Water Supply for the World's Columbian Exposition at Chicago.

Mr. Harvey's railroad career began in 1895 when he was employed by the Illinois Central Railroad Company as Instrumentman on the lake-front improvement work in Chicago. He was soon made Assistant Engineer in charge of the elevation of the St. Charles Air Line in Chicago, and later given charge of the construction of the Fort Dodge and Omaha Railway, now a part of the Illinois Central System. On the completion of this work he was made Roadmaster in charge of engineering and maintenance at Dubuque, Iowa.

In April, 1902, he left the Illinois Central Railroad Company to become Division Engineer on the Chicago Great Western Railway, with which he was connected until 1905, being actively engaged in maintenance work and the construction of various track extensions. In 1905 Mr. Harvey returned to the Illinois Central Railroad Company as Division Engineer in charge of construction of the Indianapolis Southern Railroad, from Indianapolis to Bloomington, Ind. Following this work, he became Division Engineer of the St. Louis, Iron Mountain, and Southern Railway at Little Rock, Ark.

At this time Mr. Harvey left railroad work and became associated with Mr. R. H. Elzy, of Marshalltown, Iowa, a railroad contractor. This firm handled the construction of double track on the Chicago Great Western Railway from Galena to Stockton, Ill., the extension of various tunnels on the line of the Chicago Great Western Railway, and extensive grading work in the Town of Gary, Ind. On the completion of the latter he again took up railroad work, and was placed by the Northern Pacific Railway Company in charge of station improvement at Billings, Mont. He then returned to the Chicago Great Western Railway Company as Division Engineer, and, later, became Division Superintendent, which position he held until 1911. From the latter company Mr. Harvey entered the service of the Kansas City Southern Railroad Company as Division Engineer in charge of various changes of line and grade then being carried on between Kansas City, Mo., and Port Arthur, Tex. His work with this Company being completed, he entered the employ of the Kansas City Railways Company the owner of the street railway system of Kansas City, as Chief Engineer. At the time of his death, he occupied the position of Engineer of Way and Structures with the Kansas City Public Service Company, successor to the Kansas City Railways Company.

---

[1] Memoir prepared by John V. Hanna, M. Am. Soc. C. E.

Mr. Harvey's position with the Kansas City Railways Company required him to give a great deal of study to the improvement of street railway track, and he developed several new types of construction which are now used extensively throughout the United States. He was considered an authority on this subject and has contributed liberally to various publications interested in work of this character.

He was an active member of the Engineers Club of Kansas City, and was a Past-President of that organization. He was also a member of the Standing Committee of the American Electric Railway Association and, at the time of his death, was President of the Kansas City Section of the Society. He was also a member of, and took an active interest in, the American Railway Engineering Association which, at the instigation of the late John F. Wallace, Past-President, Am. Soc. C. E., his close friend and adviser, he joined soon after its organization.

In 1892 Mr. Harvey was married to Clara Myers, a classmate at the University of Illinois, who survives him. He is also survived by a daughter, Mrs. Harriet Christie, and a son, Alfred Dallas Harvey, both of Kansas City.

He won the respect of his fellow engineers and business associates by his sense of fairness and honesty in his dealings with them. He won the affection of subordinates and equals by a sympathetic interest in their problems, and hearty co-operation in their activities, both at work and at play.

Mr. Harvey was elected a Member of the American Society of Civil Engineers on April 7, 1924.

HARRY HAWGOOD, M. Am. Soc. C. E.[1]

Died January 3, 1931

Harry Hawgood was born in Derbyshire, England, on April 28, 1853. He received his collegiate education and subsequently his engineering training in London, England. Apprenticed, as was the custom there, to one of the famous engineering firms, his training was of the most thorough and practical kind, and to the end of his life he always spoke with respect and affection of the able engineers who were his preceptors at that time.

In 1874, Mr. Hawgood was appointed to the Engineering Department of the Colonial Government Railways of the Cape of Good Hope, South Africa, and for five years he was employed on a wide range of engineering, from the design of bridges and other structures to maintenance of way and executive work. Leaving Africa with the highest commendations of his superiors, he came to the United States and entered the employ of the Chicago and Northwestern Railroad Company.

---

[1] Memoir prepared by a Committee of the Los Angeles Section consisting of C. R. Sumner, *Chairman*, and George S. Binckley, Members, Am. Soc. C. E.

In 1881, he took charge of a large party for the Union Pacific Railroad Company, making an extensive reconnaissance through Montana, Idaho, and Eastern Oregon. Those were the years during which the engineers blazed the trails followed by the great transcontinental railroads, and Harry Hawgood was one of the most daring and active of the great railroad engineers of those times. Through wild and unexplored country with the lightest of camp outfits, over treacherous rivers and unknown mountains, for two years, winter and summer, he explored and studied and mapped that vast terrain. After making the location for the Oregon Short Line from the mouth of the Boise River and along the Snake, to the mouth of Burnt River, he began the first survey ever made through the 120 miles of canyon to the mouth of Salmon River. Before this survey was made, the only knowledge of that wild stretch of river was obtained some years before through the hair-raising adventure of a party who had successfully run a small steamer through the canyon during the period of high water. For two seasons Mr. Hawgood was baffled in his undertaking by the insuperable difficulties of the terrain, but during the third year, using such scant data as were furnished by the one party which had gone through before, he built some stout boats for the expedition, and, with these as his sole resource, pushed through successfully, losing but two men as the price of a notable achievement.

Following a year of studies for the improvement of the Oregon Railway and Navigation Company's lines along the Columbia River, and in charge of the construction of the line from Hilgard to Baker City, Ore., Mr. Hawgood entered private consulting practice in Portland, Ore. In this capacity, he was consulted on many railway and other engineering projects, including the Bull Run water supply for the City, and he was Chairman of the Board which passed on the great draw-span of the Willamette River Bridge. He was Consulting Engineer to the Oregonian Railway Company, and after serving as Chief Engineer of the Portland and Willamette Valley Railroad, he became, in 1888, Resident Engineer of the Southern Pacific Lines between Los Angeles, Calif., and El Paso, Tex.

From 1888 to 1894, Mr. Hawgood carried the heavy responsibilities of this position with energy and success. This period was one of the most active in his career, for during this time he was many times confronted with conditions which required instant and correct judgment—such, for instance, as the disastrous floods which devastated Los Angeles County in 1889. Every bridge across the Los Angeles River was washed out, and only a single wire to Chicago, Ill., remained, as communication with the rest of the world; in ten days Mr. Hawgood had restored normal rail traffic. So, again, in 1891, when the Colorado River broke across the desert for the first time known in the history of that region and flooded the Salton Basin, it was Mr. Hawgood who traced its course across the desert to the break.

From 1894, Mr. Hawgood was almost continuously in private consulting practice in Los Angeles. His work covered the field of hydraulics, railway, and harbor works, and in the long fight made by the City of Los Angeles for the development of a harbor at San Pedro, Calif., his was not less than the greatest contribution. He was indefatigable in his labors in the interests of this harbor, in writing and in personal appearances before influential bodies of engineers. In April, 1896, he went to Washington, D. C., appeared before the Senate Commerce Committee, and was instrumental in the appointment of the Walker Board, which convened in Los Angeles to consider the question of the proposed harbor. Before this Board, Mr. Hawgood presented the technical side of the problem with such effect that it has been justly considered that his influence was probably more important than any other single factor.

Returning to railroad work in 1901, at the request of the late Senator W. A. Clark, Mr. Hawgood accepted the position of Chief Engineer for the location and construction of the San Pedro, Los Angeles, and Salt Lake Railroad, now a part of the Union Pacific System. On the completion of his labors he became Consulting Engineer for this road, and resumed his private practice in Los Angeles in 1905. From that time Mr. Hawgood was often consulted by the City of Los Angeles and other municipalities, was a member of the Board of Engineers Flood Control of Los Angeles County, and was tireless in his efforts to secure the building of a Union Railway Terminal at the Plaza site in Los Angeles. He designed and supervised the construction of the Huacal Dam near Nacozari, Mexico, and the Escondido Dam in San Diego County, California. His knowledge of the climatology and water resources of California was broad and thorough and his services were much sought in water litigation, not only on account of his expert command of the subject of hydraulics and water law, but because of the fact that his ethical standards and rigid integrity gave to his testimony a weight that was recognized by both Bench and Bar.

A quiet and unassuming man, with neither thirst nor talent for publicity, Harry Hawgood was none the less honored by those from whom he desired appreciation. He was a member of the Institution of Civil Engineers of Great Britain, Past-President of the Los Angeles Section of the Society, a member of the American Railway Engineering Association, an Honorary Member of the Southern California Chapter, American Institute of Architects, a member of the Arch Dam Committee, Engineering Foundation, and Chairman of the Test Dam Committee. He was a Director of the Los Angeles Chamber of Commerce from 1897 to 1898, and one of the oldest members of the Jonathan Club. To all these organizations he contributed freely of his services and support, and, in his later years, gave an increasing amount of his time and energy to research.

Such in brief outline is the record of a long, active, and successful professional life; but it is not a portrait of the man.

Harry Hawgood was of a physical type strongly characteristic of his race. Of medium height, with a fine head and brow but lightly furrowed, his face showed the strong strain of Danish ancestry which came down to him from the wild Vikings who first ravaged and then were absorbed by the England of the Ninth and Tenth Centuries. A strong chin and jaw gave more than a hint of courage and firmness. Keen but kindly grey eyes were set under well-marked brows, and the firm but humorous mouth was shaded by a moustache. Thoughtful, deliberate, well-poised, he was an unforgettable man. Naturalized in 1895, there was not a more loyal citizen, yet to the end (unconsciously perhaps) he remained an Englishman, with all the curious reserve and "stiff-necked" personal pride of the best of his race. He was indeed a fine example of the English gentleman. Of high intellectual qualities, his traditions, education, and reading gave him a breadth of culture and taste that was as admirable as it was rare. His was a questing mind, and he never ceased to learn as the busy years of his life flowed past. There was nothing neutral in his character. Behind the quiet courtesy and consideration which marked his manner, there was plenty of steel if need be—plenty of courage, moral and physical. A man of perfect intellectual integrity, he detested hypocrisy and pretense of every kind. Unobtrusively but firmly he held to his own ideals, but never tried to force them upon others. He was no missionary, no propagandist. Tolerant, philosophical, he lived his own life and let others live theirs in peace. Many were his friends but few his intimates. He felt and appreciated sympathy and understanding, yet his characteristic reserve did not encourage its expression, and in his last years even those closest to him were not allowed to walk quite by his side in the shadow that was closing in.

Mr. Hawgood was married in December, 1897, to Harriet E. McWain, a descendant of one of the old Revolutionary families of Vermont. For thirty years his life with this admirable woman was one of mutual devotion. Those were his happiest and most fruitful years. Indefatigable in his profession, devoted to his home and all it implied, the loss of his wife in February, 1927, was a blow from which he never recovered. Already advanced in years, already warned of his failing health, only those closest to him could see a little through the brave screen of unfailing good cheer and interest in his work, and know the tragedy of his irreparable loss. In spite of an indomitable spirit, however, his strength failed steadily through those few lonely years until, after a brief illness, cheerful and gallant to the last, his life came to a peaceful end on January 3, 1931.

Taken only at the full term of a long and useful life, the passing of Harry Hawgood is none the less a personal loss that will long be mourned by those who knew him.

Mr. Hawgood was elected a Member of the American Society of Civil Engineers on May 4, 1909. He served as a Director of the Society from 1918 to 1920.

ALLEN HAZEN, M. Am. Soc. C. E.[1]

Died July 26, 1930

Allen Hazen, the son of Charles Dana and Abbie (Coleman) Hazen, was born in Hartford, Vt., on August 28, 1869. He attended the schools of his native town and the New Hampshire College of Agriculture and Mechanic Arts, from which he was graduated in 1885.

---

[1] Memoir prepared by George W. Fuller, M. Am. Soc. C. E.

After completing a special course in Sanitary Chemistry at the Massachusetts Institute of Technology in 1888, Mr. Hazen was appointed, on the recommendation of Dr. T. M. Drown, then Chief Chemist of the Massachusetts State Board of Health, to take charge of the Board's Experiment Station at Lawrence, Mass. The Lawrence investigations were a classic pioneering work in the methods of purification of water and sewage from a combined engineering, chemical, and biological viewpoint and did much to direct attention to and confirm the germ theory of disease. With numerous severe epidemics of typhoid fever, this was a critical period in the development of improved water supplies for cities in the United States.

While Mr. Hazen was at Lawrence an epidemic of Asiatic cholera occurred at Hamburg, Germany, and the American public was much disturbed by the appearance of that disease on ships in New York Harbor. Under these stimulating circumstances, the investigations at Lawrence were continued and included extensive studies by Mr. Hazen of methods of measuring the physical properties of sands, gravels, and other materials, and of comparing their suitability for practical use in water or sewage filters. Widespread attention was given to the data of the Lawrence Station which was installed initially to demonstrate the feasibility of disposing of the sewage from the North Metropolitan Sewerage District of Boston on the Saugus Marshes, or otherwise by "land treatment". In fact, the Lawrence results, as contained in the State Board of Health reports for several years beginning in 1890, were regarded as epoch-making in establishing this branch of sanitary science on a sound modern basis; and this view was shared throughout the world, not only by engineers, but by medical health officers and laboratory workers in various fields.

In the design and supervision of construction of the intermittent sand filter for the Lawrence water supply, drawn from the polluted Merrimack River, the late Hiram F. Mills, Hon. M. Am. Soc. C. E., Engineer Member of the Massachusetts State Board of Health, who was assisted in that work by Mr. Hazen, availed himself of various studies and practical tests at the Experiment Station. In fact, the methods of mechanical analysis of sand developed by Mr. Hazen were at once adopted by engineers as standard and have been used almost exclusively since then.

Mr. Hazen's next work was at the World's Fair in Chicago, Ill., in 1893, where he was in charge of the disposal of sewage at the Fair grounds. Following this service he spent a year in Europe in travel and study, including courses in hydraulics and structures at the Dresden Polytechnic Institute.

Soon afterward he entered private engineering practice in Boston, Mass., with the late Albert F. Noyes, M. Am. Soc. C. E., under the firm name of Noyes and Hazen. After Mr. Noyes' death in 1896, Mr. Hazen transferred his offices to New York City, and, in 1897, was engaged in the design and construction of the water filter plant at Albany, N. Y.—the first modern, continuously-operating, slow-sand filter plant in the United States.

In 1904, Mr. Hazen formed a partnership with the late George C. Whipple, M. Am. Soc. C. E., which continued until Professor Whipple's death in 1924. In later years, there were associated with him in this engineering practice, Weston E. Fuller, Malcolm Pirnie, and Chester M. Everett, Members, Am. Soc. C. E., and Mr. L. N. Babbitt.

While in early years water purification was a major item in the practice of the firm, its activities related to all branches of engineering in the fields of water supply and water distribution, especially dams and large pipe lines, as well as valuation work for rate-making and municipal acquisition. Mr. Hazen served a large number of the prominent water departments and water companies in various parts of North America. In 1906, he advised the Water Department of Brisbane, Queensland, Australia.

He was particularly interested in hydraulics and in hydrography, and, in addition to writing many papers on sanitary subjects, made a number of valuable and practical contributions to engineering literature on the subject of run-off of streams. He also wrote several books on subjects of interest to water-works engineers and officials, including "Filtration of Public Water Supplies," "Clean Water and How to Get It," "Meter Rates for Water Works," and "Flood Flows" published in 1930. In collaboration with Gardner S. Williams, M. Am. Soc. C. E., he compiled a volume of "Hydraulic Tables," a book which is in extensive use. He was also Editor of the Water Supply Section of the American Civil Engineer's Pocket Book.

He was very active in the affairs of the Society and frequently contributed papers and discussions to its publications. In 1900, he received the Thomas Fitch Rowland Prize for his paper entitled "The Albany Water Filtration Plant,"[2] and, in 1915, the Norman Medal was awarded to him for his paper entitled, "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply."[3] He also represented the Society on various committees, several of which are included in the following: Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations (1916-1926); Joint Committee of Founder Societies Finance Committee on Joint Activities (1926-1927); Vice-Chairman, Executive Committee of the Board of Direction (1927); Honorary Membership Committee (1927); Committee on Soils (1928-1929); National Research Council (1930); and Committee on Dams (1930).

Mr. Hazen was a thorough student who acquainted himself personally with the various details of all his problems. He applied to them an unusually clear insight, and thus drew conclusions quickly and with precision. He had a remarkable aptitude for clear and concise statement both in speech and in writing, to which he added exceptional skill in putting the results of his investigations into simple and practical form so that others could readily make use of them.

The influence of Allen Hazen's leadership in advancing water-works practice during his engineering day and age was unquestionably great. That leadership was due largely to his unusual capacity for clear, straight thinking on all major elements of an engineering proposition, and to a dominating personality. In whatever he undertook he was practical, sound, progressive, constructive, and tenacious. In his death from heart disease at Miles City, Mont., while taking a combined pleasure and business trip in the West, America lost one of its greatest engineers.

---

[2] *Transactions*, Am. Soc. C. E., Vol. XLIII (June, 1900), p. 244.

[3] *Loc. cit.*, Vol. LXXVII (December, 1914), p. 1539.

Mr. Hazen was an Honorary Member of the American Water Works Association. He was also a member of many engineering and technical societies in the United States, Canada, England, and France. He received honorary degrees of Doctor of Science from New Hampshire College in 1913 and from Dartmouth College in 1917.

In 1902 he was married to Elizabeth McConway, the daughter of William McConway, one of the leading citizens of Pittsburgh, Pa. He is survived by Mrs. Hazen, four daughters, and two sons.

At its meeting on September 29, 1930, the Board of Direction of the Society adopted the following resolution:

"In the death of Allen Hazen the Engineering Profession has lost a distinguished member who gave generously of his time and energy in its advancement.

"During his 34 years of membership in the American Society of Civil Engineers Mr. Hazen served as Director for 3 years, as Vice-President for 2 years, and on many important committees.

"The Board of Direction, cognizant of his loyal and active interest in the affairs of the Society, his contributions to technical literature, and his exemplification of the highest ethical standards, desires to place on record its appreciation of his services and to pay tribute to his qualities and attainments as a man and an engineer.

"The Board hereby records its deep regret at the loss of Mr. Hazen and extends its sincere sympathy to the members of his family."

Mr. Hazen was elected an Associate Member of the American Society of Civil Engineers on June 3, 1896, and a Member on March 6, 1900. He served as Director from 1907 to 1909 and as Vice-President during 1926 and 1927.

ALBERT BANKS HILL, M. Am. Soc. C. E.[1]

DIED JANUARY 2, 1930

Albert Banks Hill and his twin brother, Arthur Bradley Hill, were born at Redding, Conn., on May 28, 1847, the youngest of seven children of Bradley Hill and Betsy (Banks) Hill. The family was of old New England stock, having descended from the William Hill, who emigrated from England in 1632, twelve years after the sailing of the *Mayflower*, and finally settled in Fairfield, Conn. It is recorded that "he was a man of note among the Colonists." One of Mr. Hill's grandmothers was a niece of Joel Barlow, LL.D., author, poet, and diplomat.

Mr. Hill attended the common schools of Redding and prepared himself for college with the aid of one term at a private school. He entered the

---

[1] Memoir prepared by E. M. T. Ryder, M. Am. Soc. C. E.

Sheffield Scientific School of Yale University and was graduated in the Class of 1869, with the degree of Bachelor of Philosophy. One year later he received the degree of Civil Engineer. During that year, in addition to studying, he acted as Instructor in Mechanics and Surveying in the Scientific School, and also began his practical professional work as Assistant on construction of water supply reservoirs for the Town of Danbury, Conn. Mr. Hill was also an Instructor at Yale for the following year, during which period he was at one time Chainman on a construction party for the New York and Boston Air Line Railroad and at another time in charge of a party on preliminary survey for the Saugatuck Valley Railroad.

These connections with the building of original railroad lines show how far back in the engineering history of the United States Mr. Hill's work began. It provides the background for the fact that, during the last years of his life, he typified to his younger colleagues the Profession of Civil Engineering, of which profession he was regarded as the Dean in the State of Connecticut. He was practically the last of that distinguished group of engineers who can never be replaced, whose practice covered nearly the entire field of Civil Engineering. Mr. Hill had a fine record of accomplishment in Railroad, Highway, Hydraulic, Structural, and Sanitary Engineering fields which one or two generations ago would have been covered to a considerable extent by a single person, but which have now expanded so that each has developed into a specialty that can well absorb the life work of an individual engineer.

In 1871, Mr. Hill gave up teaching to enter the Engineering Department of the City of New Haven, Conn., being at first in charge of survey work and then, in the following year, in charge of the Sewer Department. He continued in the City employ and from 1883 to 1892 was City Engineer of New Haven. In 1892 he took up private practice which he continued for nearly forty years, the entire span of his engineering work having covered more than sixty years.

During this period Mr. Hill was active in the civic life of Connecticut and became one of the State's most respected and useful citizens. Particularly marked was the effect of his kindliness and high standard of ethics upon the men in his own profession. As one prominent Connecticut engineer stated, he did "more than any one in this State to elevate the profession of Civil Engineering in the eyes of his fellow citizens."

Some of the works designed by Mr. Hill in Connecticut and executed under his direction as Engineer were: The swing bridge over Norwalk River, at South Norwalk; the steel arch bridges over Mill River, New Haven; over Lake Whitney, Hamden; and over Lieutenant River, Lyme; the suspension bridge over Lake Whitney (270-ft. span), for the New Haven Country Club; the stone arch bridges, East Rock Park; reinforced concrete arches in Cheshire, Hamden, Waterbury, and over Ash Creek, Bridgeport; park drives; the East Rock, West Rock, and Beacon Hill Park Drives, New Haven public park system; part of the New Haven sewerage system; sewerage systems for Danbury and Shelton, Conn.; disposal works for Litchfield, Conn.; outfall system for Greenwich, Conn.; electric railways, Norwalk to South Norwalk, South

Norwalk to Roton Point, Norwalk to Winnipauk, New Haven to Bridgeport, Bridgeport to Fairfield and Southport, New Haven to Derby, New Haven to Cheshire, Cheshire to Waterbury, Cheshire to Milldale, New Haven to Wallingford, New Haven to East Haven, and Palmer to Ludlow, Mass.; and the Manufacturers Electric Freight Railroad, New Haven.

As Consulting Engineer he designed the reservoirs, dams, standpipes, and pipe lines for the New Haven Water Company; the Bridgeport Hydraulic Company; the Birmingham Water Company; the Greenwich Water Company; the New Canaan Water Company; the Noroton Water Company; the Stamford Water Company; and the Water Department of the City of Meriden, Conn.

In 1905 he was President of the Connecticut Society of Civil Engineers and, later, was made an Honorary Member of that organization. He was a member of the following societies and clubs: Graduates Club, New Haven; Chamber of Commerce, New Haven; Connecticut Academy of Arts and Sciences; New England Water Works Association; and of the Finance Committee of the Organized Charities Association of New Haven.

Lacking a family of his own, Mr. Hill devoted almost his entire efforts to engineering work and to his friends in the Engineering Profession. He was one of only two Connecticut citizens who have served as Directors of the American Society of Civil Engineers.

The late Albert L. Webster, M. Am. Soc. C. E., a life-long personal friend, wrote of him:

"His one and only passion was devotion to honest constructive engineering work. The magnitude and importance of the task in hand had no influence on his attitude toward what lay before him for solution. His concentration was on the most faithful and best solution of his problem be it large or small. It was the integrity of his nature practically applied to daily tasks with deliberation and calm. Spectacular projects had no appeal to him. He was always busy, but never hurried, and the long list of his accomplishments bears record of unremitting industry and love of his profession. Vacations to him were but interruptions to the life he loved best, so he took only some few holidays from his office as he felt needful for recreation of body and mind and such as he took were spent in study and contemplation of the engineering work of others akin to some problems of his own or to tramping the countryside in review of possible expansion of water supply services with which he was associated.

"He looked upon engineering as a great and almost sacred calling to be devotedly upheld by serious and honest industry and integrity. Any other point of view he regarded as a traitorous denial of faith.

"His long line of Puritan ancestry with their devotion to duty and the dictates of conscience was reflected in his simple adherence to the faith of his fathers and he retained to the end membership in the Congregational Church of his native town of Redding, Conn., the town of his fathers, and while in New Haven was a consistent attendant at the Congregational Center Church on the green, whose echoes still recall the sturdy preaching of that staunch devotee of the Faith, Dr. Leonard Bacon."

At his death the editorial in the *New Haven Courier Journal*, of January 8, 1930, accurately reflected the feeling of the community. It reads as follows:

"A FAITHFUL SERVANT AND CITIZEN

"The passing of Albert Banks Hill from the active life of New Haven ought not to be lost in the mere occurrences of the day. His career as a civil engineer is solidly written into the history of New Haven both as it reflects the activities of the local government during many years and as well the activities of private interests who found his talents indispensable. But it is not his professional ability which calls from us a tribute to his memory. That lives in certain achievements of a practical and enduring character. It is to the nature of the man that we desire to pay a delayed compliment. He was a quiet spoken, modest, and sincere man of affairs. What he was called upon to do for public or private interests he did thoroughly. In that respect he was a craftsman of the old school. He threw into his work both as teacher and practitioner the joy of the task. It fed his imagination and gave a personality to his achievement. The generation in which he was educated and trained is passing and the mass principle is affecting the life of to-day. Both as a public servant and a private citizen he possessed virtues which were translated into modest but effective service."

Mr. Hill's death occurred on January 2, 1930, in the eighty-third year of his age. He was active to the last, and had planned to go to his office as usual on the day when he passed on suddenly and quietly.

Mr. Hill was elected a Junior of the American Society of Civil Engineers on February 2, 1876, and a Member on March 5, 1884. He served as a Director of the Society in 1892.

JOHN WILMUTH HILL, M. Am. Soc. C. E.[1]

Died June 22, 1930

John Wilmuth Hill, the son of Edmund J. Hill and Amanda Wilmuth Hill, was born at Covington, Ky., on October 10, 1848. He attended the public schools of Cincinnati, Ohio, from 1854 to 1863.

Mr. Hill's practical training in engineering was obtained in various machine shops of Hamilton and Dayton, Ohio, from 1865 to 1874. This early experience assisted him materially when he was called upon to design and supervise the large projects which he brought to a successful conclusion in later life. It was during this period that he was Superintendent and Designer for the firm of Barnett and Herckelrode, manufacturers of waterwheels. From 1866 to 1870 he conducted a model shop and made many models which were sent to the Patent Office at Washington, D. C.

When he was only twenty-six years old (1874), Mr. Hill was selected by the Commissioners of the Cincinnati Industrial Exposition as Chairman of a commission to conduct tests on the Corliss engines on display and to report on their efficiency in operation for purposes of giving them a much coveted award. As an appreciation of the thoroughness of these tests, the Commissioners of the Exposition awarded Mr. Hill a special complimentary Gold Medal which is now in the possession of his only surviving son, Mr. Henry C. Hill, of Cincinnati.

In 1875, the Commissioners of the Industrial Exposition again selected Mr. Hill to conduct tests on engines on display. The unbiased opinions rendered by him on these tests brought him in contact with many builders of machinery, and he was consulted on the operation and design of various plants.

---

[1] Memoir prepared by Adolph G. Wulff, Civ. Engr., Cincinnati, Ohio.

In 1878, Mr. Hill located in Cincinnati as a Consulting Engineer. During 1880 he invented and patented a system of surface condensation for pumping engines which was adopted by the Holly Manufacturing Company and others, whereby the water pumped under control was used for condensing purposes. The year previous Mr. Hill had invented an automatic grain scale which was adopted by the leading flour mills. At this time he served as one of the Commissioners of the Cincinnati Millers' Exposition, and as an interesting and unusual exhibit persuaded five of the then leading builders of automatic cut-off steam engines to exhibit their products for tests. These engines were approximately of the same cylinder diameters and piston speed. On display were the Harris-Corliss, Reynolds-Corliss, Wheelock, Babcock and Wilcox-Corliss engine, and the Brown and Buckey engines.

At the request of Mr. Harris, of the Harris-Corliss Steam Engine Company, of Providence, R. I., Mr. Hill compiled a 200-page manual for steam engineers. This was published in 1878 and had a circulation of 20 000 copies, some of which are in use at the present time.

Mr. Hill was an ardent student of the purification of public water supplies and early in his career advanced the theory that typhoid fever was essentially a water-borne disease, and the numerous tests, both chemically and bacteriologically, that he made convinced him that the typhoid rate of a city was a criterion of the purity of its water supply. He therefore applied himself diligently to investigations as to the best method of purifying the water supply of a city.

When the City of Cincinnati, in 1896, was ready to plan for the purification of its drinking water, it was with relief that the municipal authorities turned over this work to Mr. Hill who, in conjunction with the late George H. Benzenberg, Past-President, Am. Soc. C. E., of Milwaukee, Wis., and Samuel Whinery, M. Am. Soc. C. E., of Cincinnati, formed a Board of Consulting Engineers, with himself as Chairman, to report on the best method of supplying a purified water to the citizens. Many tests of water from various sources were made, but the Committee reported on a method of slow sand filtration which is now in use in this city. The plans were submitted and approved and with some modifications the works were built at a cost of $14 000 000.

In 1897-98 Mr. Hill conducted a long series of tests on the bituminous coal available for steam purposes in the Cincinnati market, for use at the waterworks. These tests were made under Galloway boilers and embraced the usual tests for commercial purposes. Fifty-five varieties of coal were tested following the procedure of Professor Walter Johnson, of the Navy Yard, Washington, D. C. Subsequently, caloric tests were made of vein samples of New River coal which were found to be equal to the famous Nixon navigation (Welsh) coal for steam purposes and gave it prompt recognition by the United States Navy and Merchant Marine for maritime purposes.

It was in 1898 that Mr. Hill wrote and compiled an octavo volume of more than 300 pages on the "Purification of Public Water Supplies" (Van Nostrand, 1898), which was considered as an authority in those days. In 1900, Mr. Hill became Consulting and, later, Chief Engineer for the improvement and filtration of the water supply of Philadelphia, Pa., a work which required his exclusive time for six years. During his stay in Philadelphia, he was called upon by the United States Senate Committee on Affairs of the District of Columbia to give evidence as to the best methods for the purification of the Potomac River water, for use in Washington and Georgetown, D. C.

On his return to Cincinnati, he was Consultant on many improvements, notably, the water-works of Hamilton, Ohio, Knoxville, Tenn., Newport, Ky., and others.

Mr. Hill was chosen in 1904, by the Towns of Malden, Melrose, and Medford, Mass., as one of the experts to determine the value of Spot Pond, as a source of supply for part of the Metropolitan Water Supply of Boston and twenty-seven other towns forming the Metropolitan System. In 1905, in association with the late Col. Alex. M. Miller, U. S. A., and Rudolph Hering, Members, Am. Soc. C. E., he reviewed the plans prepared by Morris Knowles, M. Am. Soc. C. E., for the filtration of the Allegheny, Pa., water supply. During this year he was appointed by the Sanitary District of Chicago, Ill., as one of the experts to review the chemical and bacteriological changes in the water of the Chicago Sewage Canal at Lockport, Ill. This was an action *de novo* in the United States Supreme Court by the City of St. Louis and State of Missouri *vs.* the City of Chicago and State of Illinois, to restrain the Trustees of the Chicago Sanitary District from diverting water from the Drainage Canal for sewage dilution purposes.

In 1906 and 1907 Mr. Hill served as Chief Engineer of the old Cincinnati Water-Works preceding the completion for service of the new plant. In 1909, he was selected by the City Council of Hamilton, Ohio, to make a survey and report on the cause and prevention of floods of the Great Miami River at that city. After the serious flood of the Great Miami River in March, 1913, Mr. Hill was selected by the County Commissioners of Butler County, Ohio, to make a new and more extensive survey of the cause of the flood in this county and to report on a plan of relief for the Cities of Hamilton and Middletown, Ohio.

From 1908 to 1924, the major portion of Mr. Hill's time was devoted to the valuation of properties, both public and private, and acting as Adviser in Court proceedings, in such cases as the United Gas Improvement Company, Philadelphia, Pa.; Indianapolis, Ind., Water-Works; Chattanooga, Tenn., Water-Works; Dubuque, Iowa, Water-Works; Mobile, Ala., Water-Works; Baltimore County Water and Electric Company, Baltimore, Md.; Tampa, Fla., Water-Works; Bethlehem, Pa., Water-Works; and many others, totaling in value more than $200 000 000.

Mr. Hill was an ardent student and a great help to all younger engineers with whom he came in contact. He was very active in the civic life of his community. He was a life member of the American Water Works Association (entered 1886); a sometime member of New England Water Works Association, a member of the Franklin Institute, and the National Public Health Association; a member of the Ohio State Board of Health (1910-16); President of the Ohio State Board of Health (1915-19); and a member of the Military Staff of Governor James E. Campbell (1890-91), with rank of Colonel in the State National Guard.

Mr. Hill died at his home in Wyoming, Ohio. The land on which his home is situated has been in the possession of the family of the deceased since 1797, his grandfather, Captain Thomas Wilmuth, having purchased a homestead from the original owner, John Cleves Symmes.

One of the outstanding activities of Mr. Hill's life was the organization in 1907 of the "Schoolmates Club" which brought together annually the members of his class in the Fourteenth District and First Intermediate School. The surviving members were all present at his funeral services.

The esteem in which Mr. Hill was held by the Engineering Profession was well shown by the number of its members that attended his funeral, several of whom came long distances as a mark of respect to their former colleague. It is estimated that more than 20 000 000 people have been and are benefited directly by Mr. Hill's work, and the cost of the projects which he designed would total more than $180 000 000. During his life he published several hundred reports and pamphlets on various branches of engineering, particularly the purification of water supplies, in which he was an acknowledged expert.

In 1872, Mr. Hill was married at Dayton, Ohio, to Emma Jane Walker. He is survived by his son, Henry C. Hill, who was associated with his father for the last thirty-six years, maintaining the same office at 313 Brotherhood Bank Building, in Cincinnati.

Mr. Hill was elected a Junior of the American Society of Civil Engineers on February 3, 1875, and a Member on April 5, 1876.

ALBERT HARRISON HOGELAND, M. Am. Soc. C. E.[1]

Died May 14, 1930

Albert Harrison Hogeland was born in Southampton, Pa., on January 10, 1858, the son of John and Keziah D. (Willard) Hogeland. His early life was spent on his father's farm, and during this period he attended the public schools in Southampton. He later attended High School in Philadelphia, Pa., and afterward entered Lafayette College, Easton, Pa., from which he was graduated with the degree of Civil Engineer in 1877.

Mr. Hogeland went to St. Paul, Minn., in April, 1879, and secured employment with a surveying party on the St. Paul and Pacific Railway. This railway was just being acquired by the late James J. Hill, F. Am. Soc. C. E., and was the nucleus from which the present Great Northern Railway System has grown. His initial work was as Rodman on the first extension undertaken by Mr. Hill. This was from the northwesterly terminus of the then existing line at Fisher's Landing, Minn., to Grand Forks, N. Dak.

The history of the next few years of Mr. Hogeland's life was similar to that of many young engineers of that period. He took employment wherever railway construction was active, including the following engagements: September, 1879 to March, 1880, Leveler on surveys for a projected line of railway from

---

[1] Memoir prepared by Oscar S. Bowen, M. Am. Soc. C. E.

St. Paul to Chicago, Ill.; April, 1880, to February, 1882, Leveler, Transitman, and Topographer on the location of the main line of the Northern Pacific Railway, Yellowstone and Rocky Mountain Divisions; February, 1882, to March, 1884, Assistant Engineer for the same Company on the construction of the Bozeman Tunnel; April to September, 1884, Assistant Engineer on the St. Paul, Minneapolis, and Manitoba Railway on construction in North Dakota; and September, 1884, to June, 1885, Assistant Engineer for the Northern Pacific Railway Company on construction on the Wisconsin Division.

In July, 1885, Mr. Hogeland again entered the employ of the St. Paul, Minneapolis, and Manitoba Railway Company, and remained continuously with this Company and its successor, The Great Northern Railway Company, until his death. During these years he held positions, successively, as follows: From July, 1885, to September, 1890, Assistant Engineer on construction and maintenance on lines in Minnesota, North Dakota, South Dakota, and Montana; September, 1890, to August, 1896, Engineer of Maintenance of Way, Eastern District; August, 1896, to March, 1902, Resident Engineer, Eastern District; March, 1902, to February, 1903, Assistant Chief Engineer; February, 1903, to May, 1925, Chief Engineer; and from May, 1925, to the time of his death, Consulting Engineer.

In 1929, Mr. Hogeland planned and supervised the construction of the latest extension undertaken by the Great Northern Railway Company—a branch line in Montana, extending from the Town of Saco on the main line, in a northwesterly direction, 80 miles to the Town of Hogeland, which was named for him.

It will be noted that he was connected with the first and last pieces of line construction thus far carried out by Mr. Hill and his successors in creating the Great Northern Railway System. Of his 51 years of active career, he spent 46 years with the Great Northern Company, the last 45 of which were continuous. He served as Chief Engineer for 22 years and, during this period, planned and supervised the construction of about 2 500 miles of the Great Northern System, as well as extensive betterments.

Some of the larger improvements carried out under Mr. Hogeland's supervision were: Extensive steel and concrete ore-handling docks and grain elevators, at Superior, Wis.; electrification of the old Cascade Tunnel, 2$\frac{3}{4}$ miles long, including the construction of a hydro-electric plant near Leavenworth, Wash., on the east slope of the Cascade Mountains; ocean docks and warehouses at Everett, Wash., and Vancouver, B. C., Canada; and about nine miles of timber and concrete snowsheds in the Cascade Mountains. The Cascade Tunnel and these snowsheds were abandoned in the fall of 1928 on the completion of a new tunnel, eight miles in length. He also supervised the construction of extensive terminal improvements at Seattle, Wash. These included a double-track tunnel under a part of the city, giving the Great Northern Railway an entrance to the city without grade crossings. He also planned and supervised large freight and passenger terminals at Vancouver, B. C.

In addition, his work included a number of freight terminals with the necessary buildings and other facilities; the enlargement and rebuilding of facilities at other terminals; a large number of steel bridges, including a bascule bridge across Salmon Bay, at Seattle, and lift-span bridges across the Missouri and Yellowstone Rivers. He was likewise instrumental in effecting extensive line revisions.

Mr. Hogeland was also Chief Engineer for the Glacier Park Hotel Company, a subsidiary of the Great Northern Railway Company, and supervised the extensive developments of that Company in Glacier National Park, including roads, hotels, and chalets; and one of his latest works was the construction of the Prince of Wales Hotel and other improvements in Waterton Lakes Park, located in Canada, north of and adjoining Glacier National Park.

Seldom does it fall to the lot of an engineer to assume responsibility for the creation of such extensive physical property, and to leave behind such a record of service and accomplishment. This was possible through an unusual capacity for work, persistent, untiring effort, and a great loyalty which his efficient leadership and helpful fairness developed in his associates. Also, Mr. Hogeland had the unusual privilege of close association, during a considerable portion of his career, with Mr. James J. Hill, to whose great vision and remarkable energy is due the creation of the Great Northern Railway System.

The passing of Mr. Hogeland was an occasion for deep regret on the part of a host of associates and friends from the Mississippi Valley to the Pacific Coast. His extended career through this section had given him a wide acquaintance, and one of his strongest characteristics was his ability to form enduring friendships. In his passing the world has lost a most useful citizen, the Engineering Profession, an outstanding member, and his associates, a most helpful friend.

He died May 14, 1930, while still actively engaged in his work. His health had been failing for about a year, but the severe illness which resulted in his death was of only a few days' duration.

Mr. Hogeland was a member of the American Railway Engineering Association, the Engineers' Society of St. Paul, the Minnesota Club, and the Town and Country Club of St. Paul.

He was married on January 10, 1889, to Elizabeth T. Trego, of Newtown, Pa., who, with his daughter, Mrs. John de J. Pemberton, survives him.

Mr. Hogeland was elected a Member of the American Society of Civil Engineers on May 2, 1906.

HENRY HOLGATE, M. Am. Soc. C. E.[1]

Died January 22, 1930

Henry Holgate was born at Milton, Ont., Canada, on September 14, 1863, the son of John and Jane (Browne) Holgate, who were of English descent.

---

[1] Memoir prepared by M. J. Butler, M. Am. Soc. C. E.

His early education in the public schools and in the High School in Toronto, Ont., Canada, prepared him for an apprenticeship, from 1878 to 1883, to the Northern Railway Company of Canada, under the direction of F. W. Cumberland, Chief Engineer and Managing Director.

On the completion of his service, Mr. Holgate was placed in charge of the Northern Railway and the Hamilton and North Western Railway, both of which were absorbed by the Grand Trunk Railway Company of Canada, under the general management of Sir Joseph Hickson, President of the Grand Trunk, with headquarters in Montreal, Que., Canada, to whom Mr. Holgate reported directly. During the tenure of office under Mr. Holgate, these railways were practically re-built, permanent structures being erected in place of the temporary works. The gauge of the Northern Railway was changed to the standard 4 ft. 8½ in., from the former 5 ft. 6 in., and yards and depots were gradually improved to a fairly high standard. This period extended from 1883 to 1892.

From 1892 to 1894, Mr. Holgate acted as Engineer for the Central Bridge Company at Peterborough, Ont., Canada, designing and building highway and railway bridges and various structures, including the Union Station at Toronto. During the next two years (1896 to 1898), he served as Chief Engineer and Manager of the Montreal, Park, and Island Electric Railways, a suburban system of Montreal, and as Construction Engineer for the Company on suburban electric railways.

From 1898 to 1900, he was Chief Engineer and Manager of the West India Electric Company, of Kingston, Jamaica. He developed power on the Rio Cobre, transmitted it to Kingston, built and equipped twenty-seven miles of electric railway, and also operated it. In 1900, he engaged in general engineering practice, maintaining an office in Toronto. He built the hydro-electric plant for the Canadian General Electric Company, at Peterborough, and constructed the hydraulic work for the Seven Islands Power Company, on the north shore of the St. Lawrence River, 320 miles northeast of Quebec, having previously explored the region and selected the site for the piers, power plant, etc.

From 1901 to 1911, Mr. Holgate was in partnership with Mr. Robert A. Ross, an eminent Electrical Engineer. During this period the firm built a great number of hydro-electric plants, notably for the International Nickel Company, on the Spanish River; the West Kootenay Power Company, in British Columbia; the Sherbrooke Light and Power Company, the Disraeli, the Farnham, and others. In 1907 Mr. Holgate was appointed by the Government of Canada as Chairman of a Commission to report on the causes that led to the Quebec Bridge disaster. He and his associates produced a fearless and masterly report.

When the partnership with Mr. Ross was terminated in 1911, Mr. Holgate again engaged in private practice. His next work of importance was the 180 000-h.p. Cedar Rapids Development, on the St. Lawrence River. In 1915, he prepared and presented a paper on this work[2] before the Canadian Society of Civil Engineers (now the Engineering Institute of Canada), for which he was awarded the Gzowski Medal. He also was engaged on a number of other works, his last outstanding service as a Consulting Engineer having been a report, made in collaboration with Mr. J. A. Jamieson, on the St. Lawrence Deep Waterway Project.

---

[2] "The Hydro-Electric Power Plant of the Cedars Rapids Hydraulic Manufacturing Company, at Cedars, P. Q.", *Transactions*, Canadian Soc. of Civ. Engrs., Vol. 29 (1915), Pt. 1, p. 440.

Mr. Holgate was a gentleman of courage and ability, and his death, from a sudden attack of angina pectoris, is a loss to the Engineering Profession. While he did not confine himself to hydro-electric development, he was internationally known as an expert on that subject, and was much sought as a Consultant.

He was a member of the Engineering Institute of Canada, American Society of Mechanical Engineers, and the Engineers Club of Montreal, of which he was a Past-President.

He was married at Milford, Del., on September 19, 1888, to Bessie Bell Headley who, with two sons, Henry W. and Edwin H., survives him.

Mr. Holgate was elected a Member of the American Society of Civil Engineers on October 2, 1901.

RICHARD LEVERIDGE HOXIE, M. Am. Soc. C. E.[1]

DIED APRIL 29, 1930

Richard Leveridge Hoxie, the son of Joseph and Jacqueline (Barry) Hoxie, was born in New York, N. Y., on August 7, 1844. He came of Colonial stock, both his great-great-grandfather, Lt.-Col. Gideon Hoxie, and his great-grandfather, Lieut. Peleg Hoxie, having served in the Revolutionary Army. He attended schools in New York, Pennsylvania, and Italy, from 1851 to 1858, and was a student at Iowa State University from 1858 to 1861.

At the outbreak of the Civil War Mr. Hoxie enlisted in the First Iowa Cavalry and served as Bugler, Private, and Corporal in Company F of that Regiment from June 13, 1861, to June 9, 1864. While he was still in the Army, he received an appointment as a Cadet at the United States Military Academy, at West Point, N. Y., from Iowa. He entered that institution on July 1, 1864, was graduated third in his class on June 15, 1868, and was appointed on that date as a Second Lieutenant in the Corps of Engineers, U. S. Army. He began his commissioned service with the Engineer Battalion at Willets Point (now Fort Totten), N. Y., where he remained until 1870, when he was transferred to duty on the public works at Boston Harbor, Massachusetts.

In 1872, Lieutenant Hoxie was assigned to duty under the orders of Lieut. George M. Wheeler, Corps of Engineers, U. S. Army, then in charge of the geographical survey of the United States west of the 100th Meridian; he served as Assistant on this work until 1874.

In 1874, Lieutenant Hoxie was appointed Engineer to the old Board of Commissioners of the District of Columbia and served in this capacity until 1878, when under the new District of Columbia Government he became Assistant to the Engineer Commissioner. He was promoted to the grade of Captain, Corps of Engineers, in 1882, and remained on duty as Assistant to the Engineer Commissioner of the District of Columbia until 1884. During the last two years of this service he was also on duty in connection with the extension of the Washington Aqueduct for the improvement of the water supply of the National Capital.

---

[1] Memoir prepared under the direction of Maj.-Gen. Lytle Brown, Chf. of Engrs., U. S. A., M. Am. Soc. C. E., Washington, D. C.

In 1884, Captain Hoxie was assigned to duty at Montgomery, Ala., where he remained until 1889 in charge of various river and harbor improvements in Alabama, Georgia, and Florida, and also of the defensive works at Pensacola, Fla. He was then transferred to duty at Willets Point, where he served as Instructor at the Engineer School of Application and with the Battalion of Engineers, until 1893. His next station was at Pittsburgh, Pa., where he was in charge of the improvement of the Allegheny and Monongahela Rivers until 1897. He reached the grade of Major in the Corps of Engineers in 1895.

In 1897, Major Hoxie was transferred to duty at Portland, Me., in charge of river and harbor and fortification works in Maine and New Hampshire, where he remained for about one and one-half years. In 1898 he was assigned to duty as Engineer Secretary of the Lighthouse Board, in Washington, D. C.

He served with the Lighthouse Board until 1901 when he was sent to St. Paul, Minn., in charge of the works of river and harbor improvement in that District. In 1903, Major Hoxie was transferred to Baltimore, Md. His duties in Baltimore consisted of river and harbor improvements and fortification work and, in addition, he served as Engineer of the Fifth Lighthouse District and as a member of the Board of Engineers for Rivers and Harbors. He reached the grade of Lieutenant-Colonel, Corps of Engineers, in 1904, and that of Colonel, in 1907. On August 7, 1908, while still on duty at Baltimore, having reached the statutory age of retirement—sixty-four years—he was placed on the retired list of the Army, with the rank of Brigadier-General.

General Hoxie's service to his country was practically continuous over a period of nearly half a century. It began when as a boy, at the outbreak of the Civil War, he responded to the call for volunteers. He was the last member of that group of officers of the Corps of Engineers who had active service in the field during the great conflict of 1861-65. He participated in the explorations which gave to the people of the United States the first accurate information of the territories of the Far West, a vast area that sixty years ago was practically *terra incognita.* His services as City Engineer in Washington, then at the beginning of its development, were characterized by foresight and engineering skill. Much of the sewer and water systems, and many of the streets which serve the people of the Capital City to-day, were planned and built by him more than fifty years ago.

In 1873, General Hoxie made a visit to Florida and at that time realized its advantages as a winter resort. He was a close friend of Mr. Henry M. Flagler, who later developed the east coast of Florida and built the Florida East Coast Railway.

After his retirement, General Hoxie spent part of each year in Washington, in his boyhood home in Iowa City, Iowa, and in Miami, Fla., where he maintained a winter home. He died in Miami, and his remains were brought back to Washington, where, after a simple service in his home, his ashes were placed in the Arlington National Cemetery, the last resting place of an ever-increasing company who have served their country loyally and well.

General Hoxie was married on May 28, 1878, to Vinnie Ream, the celebrated sculptress, who died on November 20, 1914, leaving one son, Richard Ream Hoxie. On April 30, 1917, he married Mae Ruth Norcross, who, with his son, survives him.

General Hoxie was a member of the Grand Army of the Republic, the Sons of the American Revolution, the Loyal Legion, and of the Metropolitan, Army and Navy, and Cosmos Clubs of Washington. At one time he belonged to the Duquesne Club of Pittsburgh and the University Club of Baltimore.

General Hoxie was elected a Member of the American Society of Civil Engineers on June 2, 1886.

WILLIAM HAUSMER HOYT, M. Am. Soc. C. E.[1]

DIED NOVEMBER 10, 1927

William Hausmer Hoyt was born at Owatonna, Minn., on October 13, 1867, the son of Americus V. and Catherine A. (Colvin) Hoyt. The family moved to Minneapolis, Minn., in 1870. He was educated in the public schools of Minneapolis, was graduated from the University of Minnesota as Bachelor of Civil Engineering in 1890, and received the degree of Civil Engineer in 1895. He was a member of the Sigma Chi Fraternity.

Mr. Hoyt spent the summer of 1888 as Instrumentman for the Chicago and Great Western Railway Company, and that of 1889 in charge of a party on general surveying for a firm in Duluth, Minn. Duluth was then in the early stages of its development into a great transportation center. To this development almost all of Mr. Hoyt's professional life was devoted.

He spent the first year after his graduation in charge of street grading, paving, and the construction of a water system and sewer system for Lakeside, now a part of Duluth; for the next two and one-third years he was in charge of the installation of machinery, and the construction of about twenty miles of line for the Duluth Street Railway Company; he then spent more than three years as Assistant Engineer of the Duluth and Iron Range Railroad Company in charge of the location and construction of lines on the Vermilion and Mesabi Iron Ranges; this was followed by six months on road construction for St. Louis County.

From June, 1897, to December, 1900, Mr. Hoyt was Junior Assistant Engineer under the U. S. Engineer Office, at Duluth, on the construction of the Presque Isle Breakwater, at Marquette, Mich. (1897); on harbor improvements at Ashland, Wis. (1898); and on the construction of the foundation for the harbor entrance at Duluth (1899 and 1900). From January, 1901, to April, 1903, he was Principal Assistant City Engineer of Duluth in charge of all construction.

---

[1] Memoir prepared by a Committee of the Duluth Section consisting of O. H. Dickerson, W. A. Clark, and W. E. Hawley, Members, Am. Soc. C. E.

In April, 1903, Mr. Hoyt began his work as Assistant Engineer of the Duluth, Missabe, and Northern Railway Company, and continued with this Company until his death. In January, 1905, he was made Assistant Chief Engineer in charge of all construction work, maintenance of way and bridges, buildings, ore piers, and coal piers. He became Chief Engineer in February, 1920, and was elected a Director of the Railway Company on July 1, holding both positions until his death.

During the years of his service, the Duluth, Missabe, and Northern Railway Company completed a great amount of construction work, building a large coal-receiving pier, three large ore piers, one of wooden construction, and two of steel and concrete, besides an extensive program of new lines, numerous bridges, and double-tracking. A large car shop and roundhouse and many other projects were also constructed.

Mr. Hoyt's ability and zeal were widely recognized in the Society, and at the time of his death he was prominently mentioned as a candidate for Director from his District. He was active in promoting the regular monthly meetings of local members of which the first was held on November 17, 1914, and from which the Duluth Section of the Society was organized on April 17, 1917. On July 15, 1918, the Section appointed him its Representative on the Committee of Development of the Society, of which he was a very active, devoted, and progressive member.

Believing with many other engineers that the profession should secure much greater public recognition, he gave much thought as to how this could be done. Two methods often have been suggested, namely, more effective organization of engineers, and the more general and active participation of the engineer in public affairs.

Mr. Hoyt was an outstanding pioneer and leader in the more effective organization of the entire Engineering Profession, and he exemplified for many years of his life, in a very remarkable way, that part of the Code of Practice of the Society which refers to the relations of the engineer with the public.

At the Annual Meeting of the Minnesota Surveyors and Engineers Society, at Duluth, early in 1912, he suggested the formation, by co-operative action, of a body which could effectively represent all the architectural and engineering organizations of the State.

In 1915, he and five other leaders in six of the engineering organizations of the State formed a voluntary body which, in 1916, became the Minnesota Joint Engineering Board officially representing these organizations. He was a leader in the work of this Board and in the planning for its development into the present Minnesota Federation of Architectural and Engineering Societies, which very efficiently represents 10 member organizations with a total of about 1 000 members of the professions in the State.

He was also active in forming the Duluth Engineers' Club, in 1918, and served five years as its first representative on American Engineering Council.

Mr. Hoyt was an active member of the American Railway Engineering Association, serving four years as Chairman of Committee VII—Wooden Bridges and Trestles. He was the first Chairman of the Duluth-Superior Joint Harbor Commission, appointed by the Governor of Minnesota, and was long active in the Duluth Chamber of Commerce. At the time of his death he was Director-Chairman heading three groups: The Legislation and Taxation Committee, the Safety Bureau, and Public Health.

He was a member of the First Presbyterian Church of Duluth and was Chairman of its Board of Trustees. He had also served as a member of the State Committee of the Young Men's Christian Association for several years.

Mr. Hoyt was an active member of all the Masonic bodies, having been a Thirty-third Degree, Scottish Rite Mason, Past Master of Palestine Lodge No. 79, a member of Duluth Commandery, Knights Templars, and a Noble of Aad Temple of the Mystic Shrine.

He was married to Jessie May Nicol, at Minneapolis, on October 3, 1895. Mrs. Hoyt, with a son, Allen N., and a daughter, Margaret Helen, survives him.

Mr. Hoyt was elected a Member of the American Society of Civil Engineers on June 30, 1911.

MARION COLUMBUS HUCKABY, M. Am. Soc. C. E.[1]

Died August 13, 1930

Marion Columbus Huckaby was born near Coushatta, La., on July 22, 1889, the son of M. P. C. Huckaby.

He attended the public schools of his native State, the Polytechnical Institute, at Ruston, La., and, in 1914, received the degree of Bachelor of Science, from the Louisiana State University. Subsequently, he took a postgraduate course at Columbia University, New York, N. Y., and, in 1917, he received the degree of Master of Science.

On the completion of his college course, at the age of 28 years, Mr. Huckaby entered the United States Army, at Fort Sam Houston, San Antonio, Tex. After a short period, he was transferred to Fort Scott, San Francisco, Calif., and, later, to Fort Ward, Washington, ranking at the close of the war as First Lieutenant, Coast Artillery. He was ordered overseas, just prior to the signing of the Armistice.

In civil life, Mr. Huckaby had the following engineering connections: The Illinois Highway Department, the Missouri Highway Department, and the Louisiana Highway Department. For a limited time, he was in private practice at Baton Rouge, La., later becoming identified as Engineer Representative of the Southern Surety Company, with headquarters at St. Louis, Mo. For more than a year previous to his untimely death, he was associated with the U. S. Fidelity and Guaranty Company, with headquarters at Oklahoma City, Okla.

Aside from his technical ability, Mr. Huckaby had a pleasing personality, and a disposition to co-operate in the betterment of the Engineering Profession and to aid in local civic affairs. He was a member of the Engineers Club of St. Louis; the American Legion; the Oklahoma City Chamber of Commerce; and the Modern Woodmen of the World. He was also a Thirty-second Degree Mason and a member of the Presbyterian Church, at Normandy, Mo.

---

[1] Memoir prepared by James C. Travilla, M. Am. Soc. C. E.

Mr. Huckaby's death was most distressing to his family, friends, and associates, as he was killed in an airplane crash at the Municipal Airport, Oklahoma City, on August 13, 1930.

On June 8, 1918, at Seattle, Wash., he was married to Urath Dorsey McCall. Mrs. Huckaby, two daughters, and a son, survive him.

Mr. Huckaby was elected a Junior of the American Society of Civil Engineers on October 9, 1917, an Associate Member on April 19, 1920, and a Member on April 21, 1925.

HECTOR JAMES HUGHES, M. Am. Soc. C. E.[1]

Died March 1, 1930

Hector James Hughes was born on October 23, 1871, at Centralia, Pa. His parents were James H. and Mary (Miller) Hughes. He was prepared for college partly at the public schools of Williamsport, Pa., partly by a tutor, and partly by his private studies. For more than two years before entering Harvard College, he worked as a clerk. His studies, which gave no indication of his coming career, were mainly the Classics, German, French, English, History, and Economics. In 1894, when he was graduated from Harvard College, with the degree of Bachelor of Arts, he received Honorable Mention in the last named subjects.

Immediately following his graduation Mr. Hughes accepted a position as a Rodman in the office of the Chief Engineer of the Town of Brookline, Mass. He worked in this capacity until the fall of 1897, by which time he was entirely sure of his profession, for he then registered in the Lawrence Scientific School of Harvard University as a third-year student in Civil Engineering, receiving the degree of Bachelor of Science in 1899. He also served as Assistant in Mechanics in 1898 and 1899. He then accepted a position in the office of the Chief Engineer of the Chicago, Burlington, and Quincy Railroad Company, in Chicago, Ill., and, in 1900, was assigned to take charge of certain pieces of construction work in Iowa. From April to August, 1902, he was employed by the American Bridge Company, at Pittsburgh, Pa.

In the fall of 1902 Mr. Hughes was appointed Instructor in Hydraulics at Harvard University and from that time made Cambridge, Mass., his home. His successive appointments in his chosen field were as Assistant Professor of Hydraulics and Sanitary Engineering, 1903 to 1908; Assistant Professor of Civil Engineering, 1908 to 1913; Associate Professor of Civil Engineering, 1913 and 1914; and Professor of Civil Engineering, 1914 to 1930. From September, 1906, to 1911, in addition to his teaching, Professor Hughes was a member of the firm of Burke and Hughes, Consulting Engineers, of Cambridge.

Professor Hughes' contributions to the science of engineering include "A Treatise on Hydraulics" (with A. T. Safford, M. Am. Soc. C. E.), "Roads and

---

[1] Memoir prepared by Theodore Reed Kendall, M. Am. Soc. C. E.

Toll Roads in America", "Highway Engineering Education", and two articles on roads for the Cyclopædia of American Government. He was a member of many scientific bodies, including the American Society for Testing Materials, the International Navigation Congress, the Society for the Promotion of Engineering Education, the Boston Society of Civil Engineers, the National Association of State Highway Officials, the American Society for Municipal Improvements, and the American Road Builders Association. He was also a Fellow of the American Association for the Advancement of Science, a member of the American Academy of Arts and Sciences, and the Harvard Engineering Society. He was a member of the Harvard Club, of Boston, Mass., of the Oakley Country Club, Belmont, Mass., the Woods Hole Golf Club, at Woods Hole, Mass., and the Kittansett Club, Marion, Mass.

Professor Hughes was a regular attendant at the Annual Meetings of the Society and made frequent trips to New York City to attend the meetings and the Annual Dinners of the Harvard Engineering Society, in the development of which he showed constant interest.

Hector James Hughes was distinguished chiefly as an administrative officer. His executive capacity was first evidenced by the conspicuous success of his administration of the Harvard Engineering Camp, at Squam Lake, N. H. This led to his appointment as its Director in 1909, an appointment which he held until the close of his life. The many students who subjected themselves to the hard work and rigid discipline of the camp, look back with satisfaction on that experience.

During the life of the co-operative agreement between Harvard University and the Massachusetts Institute of Technology, Professor Hughes, with other Harvard professors, gave instruction at the Institute. On the establishment of the Harvard Engineering School, in 1919, he was appointed Chairman of the Administrative Board and, in 1920, was appointed Dean of the School. He held this appointment until his death.

The record of Hector James Hughes as Dean is one of faithful devotion, of sanity, and of clarity of mind in meeting the problems of re-establishing and developing the Harvard Engineering School; of a quiet insistence on the maintenance of the highest ideals in the work of the School; of firmness, understanding, and sympathy in his relations with students; and of unfailing helpfulness in his contacts with his colleagues.

On April 15, 1902, he was married to Elinor Lambert, of Cambridge, who, with their two daughters, Katharine Porter and Elinor Lambert, survives him.

The following appreciation of Professor Hughes appeared in the Harvard Engineering Society *Bulletin,* for April, 1930:

"A Soldier has gone West.

"He might have spared himself and enjoyed his one recreation—golf—and remained with us for a score of years, but that was not his way. He stayed at his post until it was too late to reinforce his ebbing strength.

"Few of us have known the real Hector James Hughes until the last few years when we began to realize the burdens he had assumed. It was his aim to relieve the other members of the Faculty as much as possible of all administrative work, taking unto himself those labors that the staff might give itself unstintingly to teaching. We know that the success of the Harvard Engineering School is in large measure due to Dean Hughes' administrative zeal.

"Dean Hughes' memory will stand as a guiding beacon to all Harvard engineers, an example of determination and faithful performance of duty as he saw it."

The *Harvard Alumni Bulletin*, of March 13, 1930, carried the following words of appreciation:

"In the death of Dean Hughes, the University has lost a leader whose guiding hand was greatly needed in the further development of its instruction in engineering. Of the personal bereavement his passing brings to friends and colleagues in the University and the profession, this is not the place to speak at length. There was in him a kind of honest sweetness, a slightly quizzical kindliness and faithfulness in the performance of the day's work, that endeared him to all with whom he came into continued contact. He took his tasks and problems seriously and brought to them great clarity of mind and an essential common sense; but he had in him, nothing of the showman or the propagandist. His was a leadership that was especially of value under the circumstances which have attended the growth of the Engineering School since 1920.

"The task he faced [in re-organizing the Harvard Engineering School after 1918] had in it unusual elements of difficulty. A great co-operative plan had been abandoned because of legal difficulty under the McKay bequest. The Engineering School was thrust back into the University as an undergraduate department and it had to find itself anew. The Dean had to re-unite the Faculty that had been merged for a time into a larger and more complex organization and thus in part disrupted. The business of building technical curricula for undergraduates and yet maintaining close connections with Harvard College had to be safely and expeditiously accomplished. There were courses to be organized, men to seek, machinery to set up and rehabilitate, and students to attract without a falling off in standards. The McKay endowment gave to the School the promise rather than the reality of affluence. The whole enterprise had to be reconceived, established on a permanent and satisfactory basis, and made to commend itself by its results.

"Into the complexities of this situation Dean Hughes brought a patient, painstaking, experimental industry. The School made no dramatic moves, but it went forward quietly, steadily, and continuously. It faces now its greatest problem. Just as Harvard College was taking on a new character under the House Plan, with resulting perplexities as to the disposition of the undergraduates in engineering, the School has been robbed of a leader who had won affection, respect, and confidence in every quarter. It is hard to estimate the debt it owes to his watchfulness and insight."

Professor Hughes was elected a Member of the American Society of Civil Engineers on May 3, 1905.

JOHN EDWARD JENNINGS, M. Am. Soc. C. E.[1]

Died March 27, 1929

John Edward Jennings was born in Brooklyn, N. Y., on April 18, 1875, the son of John S. Jennings and Mary A. (Sigsworth) Jennings.

Mr. Jennings attended the Brooklyn Public Schools until he reached the age of fifteen. At this time he secured a position in the Construction Department of the Central Railroad Company of New Jersey. While so employed he continued his studies at Cooper Union, from which institution he was graduated in 1896. His efforts resulted in a rapid promotion, and when he left the employment of the railroad (in the year of his graduation from Cooper Union), he held the position of Rodman.

---

[1] Memoir prepared by a Committee consisting of Messrs. Theodore M. Avery, and R. D. Coombs, and E. M. Van Norden, M. Am. Soc. C. E., and E. J. Staubitz, Assoc. M. Am. Soc. C. E.

Subsequently, Mr. Jennings was employed in the Drafting Rooms of the Berlin Iron Bridge Company, the Elmira Bridge Company, and the Dominion Bridge Company; he also was associated with the firms of Post and McCord and Levering and Garrigues. In 1898, he entered the employ of Milliken Brothers, Incorporated, as Draftsman and Checker. His ability soon placed him at the head of a squad having complete charge of detailing many large hotels, office buildings, and similar structures. His supervision also included mill buildings of various kinds, including some of the largest sugar-mills in the Hawaiian Islands. In 1901, Mr. Jennings received further recognition by being placed in complete charge of the Designing Division of the Company. He continued in this capacity until 1906, during which period he supervised and controlled the design of many large structures. From 1906 until 1908, he held the position of Assistant Chief Engineer.

In 1908, steel towers for supporting high-tension transmission lines came into prominence. The investigations, testing, and developments of these structures were put under the supervision of Mr. Jennings. His investigations and testing of full-sized structures led to the conclusion that the accepted methods of design were unsatisfactory. He introduced many new ideas in reference to loadings, safety factors, and working stresses to govern the design. While progress has demanded some modifications, it is a tribute to his exceptional ability and far-sightedness that his fundamental principles still govern the design of many of the present-day transmission-line structures.

When radio became a commercial possibility, Mr. Jennings quickly realized the necessity of structures for supporting the antennæ. With his wide experience in the transmission field he was in an excellent position to undertake the studies of these structures. He developed a standardized design of tower which proved highly satisfactory, and is now used at many of the important wireless stations throughout the world, including many of the broadcasting stations in the United States.

In 1918 he became Vice-President and Chief Engineer for Milliken Brothers Manufacturing Company, Incorporated. While in this position he was frequently consulted by many of the large power companies and syndicates, as well as by the Government of Norway. His work in connection with the tower designs for the Norwegian Government called him from the United States in 1920 and resulted for him in a trip around the world for a study of transmission-line conditions and construction. His travels led him to Australia, where his skill made itself evident in the design of the 100-mile transmission line extending from Melbourne to the Morwell brown-coal fields. In addition to work in the transmission and radio tower field, he was also responsible for the design of towers used for supporting the cableways in Colombia.

A few of the outstanding operations with which Mr. Jennings was connected were: Transmission towers for the Great Western Power Company; the

Southern California Edison Company (Big Creek Line); the Utah Power and Light Company; the Pacific Gas and Electric Company; the Montana Power Company; Southern Power Company; the American Gas and Electric Company; the Nevada-California Power Company; the Ebro Power Company, in Spain; Victoria Falls, South Africa; the Norwegian Government; the Nipon Electric Power Company; the São Paulo Tramway, Light and Power Company, in Brazil; and many others.

He was also responsible for major engineering work in connection with the electrification of the South African Railways; the Great Indian Peninsular Electrification Scheme; the Perak River Hydro-Electric Development; and the Central Scotland Electricity Development. Other important work under his charge included the Gammarra Cucuta Cableways, Cable del Norte, and Cable del Choco, all of which are in Colombia, South America.

Mr. Jennings was held in high esteem by his associates and employees. His fine character and ability resulted in a world-wide circle of friends. His passing has left a deep sense of loss to those who have been privileged to be associated with him in any way during his career. He was an active member in the Brooklyn Engineers' Club until his work required extensive travel abroad. He was also a member of Kane Lodge, F. and A. M., as well as the Royal Automobile Club of London.

Mr. Jennings was elected an Associate Member of the American Society of Civil Engineers on November 30, 1909, and a Member on December 3, 1913.

PETER JUNKERSFELD, M. Am. Soc. C. E.[1]

DIED MARCH 18, 1930

Peter Junkersfeld was born at Sadorus, Ill., on October 17, 1869. He was graduated from the University of Illinois in 1895, with the degree of Bachelor of Science in Electrical Engineering. His scholarship was excellent and he was made a member of Tau Beta Pi, the Engineering Honor Fraternity, Editor of *The Technograph,* and was generally prominent in student activities. In 1909, after presenting his thesis, he was awarded the degree of Electrical Engineer. From 1911 to 1913 he served as President of the University of Illinois Alumni Association.

Subsequent to his graduation Mr. Junkersfeld went to work in Chicago, Ill., for the Chicago Edison Company (now the Commonwealth Edison Company) with which he occupied a very insignificant position. He later became identified with the Engineering Department and was a Mechanical Engineer. Afterward, as Electrical Engineer, he was Assistant Vice-President in charge of engineering and construction.

The Commonwealth Edison Company, being a pioneer in the development of the steam turbine for use in central stations, sent Mr. Junkersfeld to England, in conjunction with the firm of Sargent and Lundy, to study the mechanical features of the steam turbine. He was closely identified with this great development from its very beginning in the United States until the end of his activities.

---

[1] Memoir prepared by M. J. Whitson, M. Am. Soc. C. E.

When the Officers' Reserve Corps of the Army was organized Mr. Junkersfeld was one of the first to be commissioned as Major in the Army Engineers Corps and during the emergency in 1917 when the United States entered the World War, he was one of the first to be called into active service. He was assigned to the Construction Division of the Army, an emergency organization operating directly under the Secretary of War for the construction of camps, cantonments, embarkation camps, and army bases and terminals. He was promoted to the rank of Lieutenant-Colonel and, later, to that of Colonel. In August, 1918, he was awarded the Distinguished Service Medal for his services.

During his two years' service in the Army, due to his training, his superior judgment, his great energy, and lovable character, Colonel Junkersfeld was able to perform a service in the Construction Division which perhaps should be credited as the greatest performed by any single officer. The work of this Division in providing facilities and quarters for the training of the troops and for the assembling and expediting of supplies, was very great, and Colonel Junkersfeld was called upon to give the best that was in him in these emergencies.

In 1919, he was selected as Engineering Manager for Stone and Webster and was made Vice-President of the Division of Engineering and Construction. Two years later, with Mr. W. McClellan, he organized the engineering firm of McClellan and Junkersfeld. This firm did a large engineering and construction business, among its projects being the $35 000 000 Cohokia Power Plant, at East St. Louis, Ill.

During the last two years of his life, in addition to his association with McClellan and Junkersfeld, Colonel Junkersfeld was Vice-President of the Stone and Webster Engineering Corporation.

Colonel Junkersfeld was a Fellow of the American Institute of Electrical Engineers; a member and Past Vice-President of the Western Society of Engineers; President of the Association of Edison Illuminating Companies, in 1916-1917; and First President of the Construction Division Association. He was a member of the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, the Franklin Institute, the National Electric Light Association, and the American Electric Railway Association, as well as of the following clubs: The Chicago Athletic Club, the University Club of Chicago, the Illini Club of Chicago and New York, the Engineers' Clubs of Chicago and New York, the New York Lawyers Club, and the Scarsdale Country Club, at Scarsdale, N. Y.

He was the author of many technical papers. Among these is included "General Review of Current Practice in Steam Generating Stations", of which George A. Orrok, M. Am. Soc. C. E., was co-author, and which was presented at the first World Power Conference in London, England, in 1924.

Peter Junkersfeld, born in the heart of the Middle West of sturdy and substantial parentage, learned the principles of perseverance and honor on an

Illinois farm. He sought higher education at the University, at Champaign, Ill., and afterward during his entire life he was identified with its activities and those of its alumni organizations. When his life was ended, the President and Deans of the University assembled as honorary pall bearers when he was taken to his final resting place at St. Mary's Cemetery, in South Champaign.

During his entire life Colonel Junkersfeld was extremely fond of engineering. He pursued and developed its technical phases, and applied it in its general scope to the large business enterprises with which he was connected.

Even above his love for his profession was his spirit of responsibility as a citizen. As stated, he left his responsibilities with the Commonwealth Edison Company to help the United States when the great emergency arose in 1917. Serving without personal consideration, he gave himself completely to the service of his country. His character was without blemish. To know him was to be his friend—he had no enemies.

On June 19, 1901, he was married at Champaign, Ill., to Anna C. Boyle, who, with two daughters, Florence Rita and Mary Josephine, survives him.

Colonel Junkersfeld was elected a Member of the American Society of Civil Engineers on June 16, 1919.

THOMAS PEARSON KINSLEY, M. Am. Soc. C. E.[1]

DIED SEPTEMBER 23, 1930

Thomas Pearson Kinsley was born in Manchester, N. H., on October 11, 1845, the son of Benjamin and Eliza Ann (Pearson) Kinsley. His early education was received in the public schools of his native city. At the age of thirteen years he entered the Engineering Department of the Portsmouth and Concord Railroad Company on the preliminary surveys for changes of its lines, and continued with the road during its construction. He remained with the Chief Engineer of the Concord Railroad Company (which had absorbed the Portsmouth and Concord Railroad Company) for about six years, on the construction of its various lines, and on city surveys. During this period he rose to the grade of Transitman and Leveler.

In 1864 Mr. Kinsley entered the Chandler Scientific Department of Dartmouth College, with advanced standing, so that he was graduated in two years (1866) with the degree of Bachelor of Science. During two vacations from Dartmouth, he acted as Leveler on the preliminary surveys for two railroad lines.

After his graduation Mr. Kinsley was employed in August, 1866, as Rodman by the Brooklyn (N. Y.) Park Commission, but was promoted in the following month to be a Leveler on the First Division under the late Wilson Crosby, M. Am. Soc. C. E. In the spring of 1868, Mr. Kinsley was promoted again to be Assistant Engineer in charge of the construction of Fort Greene Park, remaining with this work until it was suspended in November, 1869.

---

[1] Memoir prepared by Herbert C. Keith and Edward Gagel, Members, Am. Soc. C. E., and Robert Ridgway, Past-President, Am. Soc. C. E. Mr. Gagel died on February 11, 1931.

In later years it was a great pleasure to him to recall his experience on this work.

After an interim of nine months, during which he was employed by the Peteler Portable Railroad Company as Engineer, General Foreman, and Paymaster, Mr. Kinsley returned to the Park Commission as Division Engineer in charge of the construction of the East (or Jamaica) Parkway, of Douglas and Degraw Streets, the widening and extending of Washington Avenue, and the improvement of the East Side Lands.

In 1876, he left the employ of the Brooklyn Park Commissioners to become Chief Engineer of the Brooklyn, Flatbush and Coney Island Railroad (later known as the Brighton Beach Line), on its construction from the Brighton Beach Hotel, at Coney Island, to a junction with the Long Island Railroad at Atlantic Avenue near Flatbush Avenue, in Brooklyn. Recognizing the danger of a stairway in a crowded public passage in case of a panic, Mr. Kinsley was led to question the advisability of using stairs for the approach to the tracks at the Atlantic Avenue Terminal. Later, in 1883, his fears were justified by an accident of this nature on the Brooklyn Bridge, resulting in a serious loss of life. Crude experiments with a drawing-board supported by blocking at one end to give different grades, led to the construction of a ramp with an 8% grade, which grade later became a recognized standard for footwalk ramps where no special non-slip pavement was used. It is supposed, also, that it was on this road, under Mr. Kinsley's direction, that concrete was used first for the rim, track, and center of a railroad turn-table.

After the completion of the Brighton Beach Line in 1879, he was engaged for several years on railroad construction in New York State (the Delaware, Lackawanna and Western Railroad), Delaware, Maryland (the Western Maryland Railroad), and in South Carolina, New Hampshire, and Massachusetts (the Massachusetts Central Railroad). He was a member of a firm that rebuilt fifty miles of the Chesapeake and Ohio Canal, and, in 1890, constructed about thirty miles of electric street railway in Baltimore, Md.

For about two years, from May, 1900, to 1902, Mr. Kinsley was Engineer for L. B. McCabe and Brother, Sub-Contractors for 3½ miles of New York City's first Rapid Transit Subway (Sections Nos. 13 and 14, from 133d Street to Hillside Avenue), and he continued on this work for a number of years following the sub-contractor's retirement from it. The first actual construction work for this pioneer subway except for certain auxiliary structures like sewers, etc., was on this Section No. 13. Mr. Kinsley was himself the subcontractor for the concrete foundations of a mile of elevated structure connected with that subway. In 1910 and 1911, he was Superintendent for the Bonticou Stone Company which operated a quarry on the slope of the Shawangunk Mountains for furnishing the stone required for the construction of nine miles of the Catskill Aqueduct.

From 1911 to 1923 (when he retired from the active practice of his profession), he was Contractor and Engineer on landscape work in beautifying numerous private estates—largely in the vicinity of Cleveland, Ohio.

Mr. Kinsley was married on November 16, 1871, to Marion O. Blake, of New York, N. Y., who, with their two daughters, Mrs. F. Taylor Woodruff, of New Haven, Conn., and Mrs. Edwin L. Findley, of Cleveland Heights, Ohio, survives him. After his retirement, he and Mrs. Kinsley made their home with the Woodruffs in Leroy, N. Y., and in New Haven.

He was much interested in the promotion of knowledge of United States history; this led him to join the Brooklyn, Maryland, and New Haven Historical Societies during his residence in those several localities. Mr. Kinsley was a member also of the New York Railroad Club, the American Forestry Association, and the National Geographic Society. He was a member of the Phi Zeta Mu Fraternity (now Sigma Chi) of Dartmouth College.

He was an earnest Christian, being at the time of his death a member of the Euclid Avenue Congregational Church, of Cleveland, where formerly he had made his home for several years. His was a Christianity that made his life a benediction to all with whom he came in contact, whether in business or in social life. The affection for him that is shown by those who have worked under, beside, or over him, and by those who knew him only after the close of his active engineering life, is wonderful. Ample confirmation is found for the testimony given in the weekly calendar of the Euclid Avenue Church after his death: "He brought to us a life singularly fine, generous, and inspiring. 'So he passed over, and all the trumpets sounded for him on the other side' ".

One of his former engineering associates writes,

> "He was a man of most likable personality and very enthusiastic about his work; and his ability commanded the respect of his associates. It is hard to measure the value to the world of such a man."

To one who knew him only after he said that he had ceased his usefulness, the secret of the affection shown for him is plain—he loved others; they could not do otherwise than love him. "Ceased his usefulness?" The usefulness of such men never ends.

Mr. Kinsley was elected a Member of the American Society of Civil Engineers on February 5, 1873; he was the senior member of the Society for several months prior to his death on September 23, 1930.

GEORGE CHARLES KREUTZER, M. Am. Soc. C. E.[1]

Died November 23, 1929

George Charles Kreutzer was born on a ranch in Douglas County, Colorado, on March 6, 1884. He was the son of Edward D. and Jane Eliza (Keppel) Kreutzer. His father came to America from Germany in 1867, remained in New York and Pennsylvania for a few years, and then, in 1874, with his wife, a native of England whom he had married the preceding year, moved to Colorado where he finally established himself on a ranch in Douglas County about ten miles west of Sedalia.

---

[1] Memoir prepared by Elwood Mead and R. F. Walter, Members, Am. Soc. C. E.

Mr. Kreutzer's boyhood days were spent in hard work on his father's ranch where he gained that background of practical experience in agriculture, irrigation, and lumbering, which was to prove so valuable to him in later years.

He attended Grammar School at Sedalia and High School in Castle Rock, Colo. He entered the Colorado Agricultural College at Fort Collins, Colo., in 1903 and was graduated in 1908, with the degree of Bachelor of Science in Engineering, having taken his major work in civil and irrigation engineering. He worked his way through college under great difficulties, but even so, he made an excellent scholastic record and was an outstanding leader in all student activities. In 1922, the degree of Master of Science was conferred upon him by his Alma Mater.

On leaving college in 1908, Mr. Kreutzer's first employment was as Special Field Agent, Bureau of Irrigation Investigation, United States Department of Agriculture. Although he only occupied this position for a few months, it was the beginning of a useful career in the public service, both State and National, largely in irrigation development and land settlement problems.

From September, 1908, to April, 1910, he was employed by the Colorado Experiment Station, and was engaged in the studies of practical irrigation problems, animal husbandry, and agronomy. In April, 1910, Mr. Kreutzer again entered the employ of the United States Government for a brief period, this time with the Bureau of Reclamation, Department of the Interior, as Assistant Superintendent of Irrigation, on the Shoshone Project, in Wyoming, where he supervised the operation of the Garland Division during the summer and had charge of all survey parties during the fall and winter. He continued in this position until January, 1911.

In 1910, Elwood Mead, M. Am. Soc. C. E., Chairman of the State Rivers and Water Supply Commission of the State of Victoria, Australia, was visiting the United States. Among its other duties, this Commission operated all irrigation and water supply works serving the farming districts. While in the United States Dr. Mead asked Dr. Charles A. Lory, President of the Colorado Agricultural College, to suggest some one qualified to serve as Superintendent of one of the Australian projects, the requirements for the position being marked executive ability combined with good engineering and agricultural training and experience. Mr. Kreutzer was recommended by Dr. Lory and, as a result, he accepted an offer made by the Commission and early in 1911 was appointed Superintendent of the Cohuna Irrigation District in Northern Victoria, Australia. This assignment, which first brought Mr. Kreutzer in contact with Dr. Mead, was the beginning of an enduring friendship and professional relationship. Mr. Kreutzer's success in this work was so marked, and the progress of the District under his administration so satisfactory, that he was later transferred to the Central Office in Melbourne, Victoria, Australia, and made a general Economic Adviser in the creation of new projects and the reconstruction of old ones. He continued in this work until 1916 when, owing to the interruption in irrigation development in Australia, due to the World War, he resigned his position and returned to the United States.

In July, 1916, Mr. Kreutzer was appointed Agricultural Adviser of Kern County, California, as a part of the Agricultural Extension Service of the University of California, in co-operation with the U. S. Department of Agriculture. In this work he gave special attention to co-operative marketing and war production. His duties were later enlarged to include the supervision of county agents engaged in similar work in Kern, Kings, Tulare, and Fresno Counties. In 1918, the Legislature of California passed the State Land Settlement Act, to be under the control of a commission of which Dr. Mead was Chairman, he having returned to the United States in advance of Mr. Kreutzer. This Act had for its purpose the creation of planned farming communities of home owners organized to co-operate and to have the benefit of long-time payments and low interest rates in the purchase and improvement of their farms. Mr. Kreutzer was made Superintendent of the Durham Settlement, the first to be established under the Act, and remained in this position, which he filled with great fidelity, until 1924.

Continuing a professional association of many years' standing, Mr. Kreutzer followed when Dr. Mead was appointed by the President, in the spring of 1924, as Commissioner of Reclamation, U. S. Department of the Interior. Concordantly, in June of that year Mr. Kreutzer was appointed Chief of the Division of Reclamation Economics, under Dr. Mead. In this position, which he held until his death, he was untiring in his efforts to improve the economic status of the settlers on the Federal Irrigation projects.

He was married on March 4, 1914, in Melbourne, Australia, to Dorothy McFarlane, the daughter of an English clergyman. They had two children, William Elwood and Adelaide Dorothy, both of whom, with his widow, survive him. Also surviving him are one brother, Mr. William R. Kreutzer, and three sisters, Mrs. Minnie Cell, Mrs. Bessie Grandy, and Mrs. Agnes Klant.

Mr. Kreutzer brought to all his work the influence of a remarkable personality. In it were combined warm human sympathy, and a delightful sense of humor, together with unusual clear business sense. He was a man of much natural ability, fine personality, and extraordinary skill in dealing with people of all walks of life. He was honest, gracious, frank, and generous, loved by all who knew him for his sterling qualities of character, and respected and admired by all his co-workers for his outstanding ability and fairness.

Mr. Kreutzer was elected a Member of the American Society of Civil Engineers on January 16, 1928.

KING YANG KWONG, M. Am. Soc. C. E.[1]

DIED OCTOBER 19, 1929

King Yang Kwong, better known among his Chinese friends under the name of Kwong, Sun-Mo, was born at Nanhai District, Kwangtung Province, South China, on February 8, 1863. He received his early education in his native town and was sent to the United States in 1874, when he was only eleven years of age. He was one of the first thirty students who constituted a group known as the First Chinese Educational Mission, sent by the Imperial Chinese Government to the United States. His first few years in America were spent in Washington, D. C., where he received his preparatory education under the guidance of the veteran Chinese educator, Dr. Yung Wing, who was the Chinese Minister to America at that time. He then entered Williston Seminary at Easthampton, Mass., and in 1880, following his graduation from this school, Mr. Kwong was admitted to the Massachusetts Institute of Technology. He studied Civil Engineering there for two years, but was recalled to China by his Government in 1882, together with 119 other Chinese youths who were studying in America at that time.

---

[1] Memoir prepared by Pao-Ling Yang, M. Am. Soc. C. E.

The first position Mr. Kwong held upon his return to China was that of Assistant Engineer to the Kaiping Mining Company, at Tangshan, which Company has since been re-organized under the name of the Kailan Mining Administration, the largest coal mining company in the East to-day. In 1886, he was appointed by the Imperial Government to be the Assistant Engineer in charge of the construction of the railway from Shukochwang to Lutai, which line has since been extended to become the Peking-Mukden Railway. Mr. Kwong had the honor, therefore, of being the first Chinese railway engineer. On the completion of the section mentioned, Mr. Kwong was transferred to the position of Maintenance Engineer of the Lutai-Kuyeh Section of this railway. In the spring of 1897, he was sent to the outside of the Great Wall to take charge of the construction work on the Lienshang-Chinhsien Section. The Nüerh Ho Bridge on this section, consisting of sixteen 60-ft. spans, was built by him.

In 1900, the Boxer trouble broke out in China, and the whole line from Yinkow to Peking was seized by the foreign troops. As a result of this, in 1901, Mr. Kwong left the railway for Shanghai to become the Assistant Engineer of the Pinghsiang-Liling Railway, a short line, 60 miles long. On the completion of this line in 1902, he returned to the Peking-Mukden Railway to supervise the construction of the Koupangtse-Hsinmingfu Section near Mukden. In 1904, he was given charge of the construction of the Hanku Bridge across the Peitang River. The width of this river was approximately 500 ft., with a depth at low tide of about 23 ft., and at high tide of about 30 ft. The bridge consisted of three spans of steel trusses, two of 200 ft. and one of 90 ft. The two abutments and one pier were built on wooden piles.

In the autumn of 1905, Mr. Kwong was appointed District Engineer to the Peking-Kalgan Railway. His district extended from Ching Ho to the north of Pataling, the most difficult section of the whole line. Before he completed this work, however, he was transferred to become the Chief Engineer of the Canton Section of the Canton-Hankow Railway. At Canton, he met with innumerable difficulties. He had to solve not only the problems due to the sandy nature of the embankment and flood difficulties of the North River but, at the same time, he was forced to fight against the superstition of the people, and this, in addition to the training of workmen who were without any experience in railway construction. Later, the funds were exhausted, and the work had to be suspended. The line was completed to Shaokwan, a distance of 140 miles, leaving the remaining 70 miles to the southern border of Hunan Province unfinished as it still remains.

The most important works on the Canton-Hankow Railway were the bridges at Yintak and Shaochow. The piers of these bridges reached a depth of more than 30 ft. below the river bed. The Yintak Bridge was 300 ft. in length and the Shaochow Bridge, 700 ft., composed of spans of 100 ft. each. Mr. Kwong also established a workshop at Hwangsha for the purpose of erecting and repairing cars and locomotives. On the river bank in front of the locomotive shed a wharf was built, and a crane of 25-ton capacity was installed for the loading and discharging of cargo for steamers. The equipment of the workshop at Hwangsha was considered the most up to date of its kind in China at that time, for not only could repair work be done there, but cars could be built for the railway. The cost of construction for the whole line compared favorably with other lines in the Southern Provinces of China at that time.

In 1911, Mr. Kwong was transferred to the North and appointed Chief Engineer of the Peking-Suiyuan Railway. This road was an extension of the Peking-Kalgan Line, but owing to lack of funds it was not completed until 1921. Its total length was 204 miles and, later, a further extension of 90 miles was built to connect it with the Town of Paotou on the Yellow River. The building of the railway around the City of Peking was also done by Mr. Kwong at that time.

During the construction of the Peking-Suiyuan Line, the most difficult work experienced was the rock excavation in the mountains, in order to provide the necessary grades for the railway, and the building of the bridge at Tatungfu. This bridge comprised eighteen 100-ft. span steel girders, the rail level being about 55 ft. above the river bed. While acting as Chief Engineer of the Peking-Suiyuan Line, Mr. Kwong held concurrently the positions of Locomotive Superintendent and Works Manager to look after the building and repairing of locomotives and cars, and, during 1914 and 1915, he was also the Associate Managing Director of that line.

In his position as Locomotive Superintendent, he improved the locomotives to meet the requirements of the heavy freight traffic on the Peking-Suiyuan Railway. Air-brakes were installed on the line to increase safety while passing through the Pataling Pass. In the workshops, motive power was increased for use in repairing the rolling stock. The question of coal and water was solved by building a branch line of 14 miles to the Kouchuan Mines, from which the Tatung coal was carried to the markets. The line itself has been well supplied with this fuel since 1916.

In 1917, a disastrous flood occurred in Chihli Province, which affected seriously the Northern Section of the Tientsin-Pukow Railway. For ten miles near Tientsin, the line was entirely submerged. At this time Mr. Kwong was transferred to the position of Chief Engineer of the Northern Section. He at once took the matter in hand, and, in a short time, the railway was restored to good condition.

The construction of the entire line of the Peking-Suiyuan Railway was completed in 1920, and the increased amount of freight traffic rendered it necessary to increase the number of trains correspondingly. The question of power then arose, and Mr. Kwong introduced the use of heavy locomotives for the Pataling Pass. He was re-appointed Chief Engineer and at once started work on the strengthening of the bridges in the Pass in order to meet the requirements of the new heavy locomotives which are the largest of their kind in the Orient.

Owing to political changes as a result of the Chihli-Mukden War in 1921, Mr. Kwong retired from active service until 1929 when His Excellency, Sun Fo, Minister of Railways, appointed him Technical Adviser to the Ministry. He had served on the railways for more than thirty years and was the first engineer to enter the Chinese railway service. From a position of Assistant Engineer, he gradually rose through long years of hard work and experience until he reached the highest position in the Chinese Government Railways. He took great interest in his work and devoted himself entirely to his profession for which he earned the respect of his friends and associates.

Mr. Kwong was the first Chinese engineer who joined the American Society of Civil Engineers and had the honor of being the first President of the Association of Chinese and American Engineers. He held the rank of Expectant Taotai during the late Ching Régime and was decorated with Third-Class Chiaho, Third-Class Paokwong-Chiaho, and Fourth-Class Wenhu Medals by the Chinese Republican Government in recognition of his services. He died of heart trouble in Peking at the age of 66, and is survived by his widow, a son, and a daughter.

In writing of him, J. A. L. Waddell, M. Am. Soc. C. E., states:

> "Mr. Kwong was one of the truly great men of China, and was affectionately termed by his brother engineers 'The Grand Old Man of the Engineering Profession in China.' All who knew him intimately loved him, and every one acquainted with him held him in profound respect, not only because of his genial, kindly nature, but also on account of his noble qualities and high ideals in relation to both professional and social life. He was a true friend, an engineer of wide renown, a man of unusual intelligence, broad vision, and sound judgment, a most interesting conversationalist, and a highly polished Chinese gentleman—than whom no finer type of man can be found anywhere on earth.
>
> "His passing is a distinct loss to the Chinese Republic; for, although failing health for several years had interfered with his active usefuless, he was always ready to aid the younger men of the profession with sound and kindly advice and to give counsel to the country's rulers whenever so requested."

Mr. Kwong was elected a Member of the American Society of Civil Engineers on April 1, 1908.

EUGENE McLEAN LONG, M. Am. Soc. C. E.[1]

DIED JUNE 23, 1928

Eugene McLean Long was born in Bedford County, Virginia, near Big Island, on April 13, 1868. His father was Gen. Armistead Lindsay Long, a Confederate officer, who was Military Secretary to Gen. Robert E. Lee, and who was also the author of the "Memoirs of Lee". His mother was Mary Herron Sumner, a daughter of Maj.-Gen. E. V. Sumner, of the Federal Army.

---

[1] Memoir prepared by J. C. Meem, M. Am. Soc. C. E.

It is thus seen that, like many men of his generation, he came of stock that gave allegiance to both sides in that great upheaval of 1861, and it is not too much to say of McLean Long (or "Mack" as he was called by his intimates and friends), that he inherited and possessed many of the best qualities of the Puritan on the one hand, and the Cavalier on the other. Of fine physical appearance, he was also a fine upstanding exponent of the theory that a sound mind is housed in a sound body.

After preparation at home and at local schools, Mr. Long entered the University of Virginia, from which he was graduated in 1889, with the degrees of Bachelor of Science and Civil Engineer. The first eight years after his graduation were spent largely in the United States Geological Survey and in land and town-site irrigation work, except that during the last two years of that period he was Professor of Civil Engineering at Columbian University, Washington, D. C., being engaged between times in the private practice of engineering. For the next ten years his bent turned more toward steel inspection and erection, and he was employed at various times by the United States Government, the Baltimore and Ohio and the Pennsylvania Railroad Companies, and by New York City and various other municipalities, including Larchmont and Syracuse, N. Y. For the latter city he worked extensively on street and sewerage problems. In 1909, he became a member of the firm of Long and Miller, which firm was engaged mainly in general engineering and the design, as well as the construction and inspection, of large buildings, bridges, etc.

A contract having unusually interesting features was that of The Narrows Siphon constructed under the direction of and for the Board of Water Supply of New York City, in 1914 and 1915. The firm of Long and Miller acted as Engineers for the Merritt and Chapman Company, in laying a 30-in. pipe in a dredged trench across The Narrows. It was on this contract, at the westerly end of the work, that master-piling was first used in connection with steel sheet-piling. Here, also, the discovery was made, and the invention perfected, that steel can be burned or cut under water. The other undertakings designed or constructed, or both, by this firm, include a bridge over the Passaic River; a plant for the International High Speed Steel Company of New Jersey; bridges and piers for the Gulfport and Western Railroad Company, in Mississippi; street and sewerage system for Fort Slocum, New York, etc.

In 1917, Mr. Long resumed practice in his own name, and designed the steel plant for Halcomb and Davidson, Detroit, Mich., and a floating dry dock for the United States Shipping Board. He was also Engineer in charge of construction and maintenance of properties for several estates and for the Munson Steamship Company. In connection with foundation and other work for Swift and Company, and for Armour and Company, the writer was associated with Mr. Long on one special case and was greatly impressed with the painstaking care for detail as well as the forceful way in which he met and handled his problems.

In April, 1921, he was married to Mary Florence Gregory, of Skaneateles, N. Y. They had no children. His widow survived him only a short time.

Mr. Long was a member of the Engineers' Club and of the Society of the Virginians in New York City. His death occurred at the age of 60, at his home at Port Washington, Long Island, on June 23, 1928.

As life is measured to-day, Mr. Long was a comparatively young man; in fact, his span was ten years short of the three-score and ten allotted by the Psalmist; but measured by events and achievements he had lived a life of much greater length. Paraphrasing the words of Mayor Gaynor, as he sailed away from New York City to his death, "I have been Mayor," Eugene McLean Long could have looked back over his own constructive life and said, "I have been an Engineer."

Mr. Long was elected an Associate Member of the American Society of Civil Engineers on June 1, 1898, and a Member on July 6, 1920.

HENRY HOWELL LOTTER, M. Am. Soc. C. E.[1]

Died September 9, 1930

Henry Howell Lotter was born on September 3, 1875, in Moberly, Mo., the son of Julius H. and Charlotte (Howell) Lotter. He was of Welsh and French-German descent.

Mr. Lotter attended the public schools of his native town and following graduation from the Moberly High School, he entered the University of Missouri at Columbia, Mo. While a student at the University he was very prominent in campus activities. He held the rank of Captain in Company A in the Missouri State Military School, which was the student military organization in the University at that time. He was a member of Tau Beta Pi, honorary engineering fraternity, and of Sigma Chi, being one of the charter members of the Missouri Chapter. He was also a member of Theta Nu Epsilon. He was graduated from the University of Missouri in 1897, with the degree of Bachelor of Science in Engineering. Two years later he received his Civil Engineer Degree from the same institution.

From September, 1897, until January, 1900, Mr. Lotter was employed by the United States Deep Waterways Commission, working throughout a large part of New York State and in the Detroit, Mich., Office, in the capacities, first, of Rodman, then of Recorder, then of Instrumentman, and, lastly, of Assistant Engineer.

From January, 1900, until June, 1901, he served as an Engineer on the United States Isthmian Canal Commission. During this time he worked for six months on surveys in Panama, with headquarters at Colon. One year of this time was spent on work in Washington, D. C., preparing estimates upon the basis of which the United States Government acquired the Panama Canal.

From September, 1901, until January, 1902, he was Principal Assistant Engineer for the Louisiana Purchase Exposition Company. This work consisted chiefly of preliminary surveys in preparation for the World's Fair Exposition which was held at St. Louis, Mo., in 1904.

---

[1] Memoir prepared by C. W. Brown, M. Am. Soc. C. E.

From January, 1902, until October, 1904, Mr. Lotter was employed as First Assistant Engineer on the colossal double-track railroad bridge constructed by the Southern Illinois and Missouri Bridge Company, over the Mississippi River at Thebes, Ill. After the completion of his work on the Thebes Bridge, Mr. Lotter accepted a position as Assistant Engineer to Ralph Modjeski, M. Am. Soc. C. E., on the Vancouver-Portland double-track draw-bridges over the Columbia River and Oregon Slough, at Vancouver, Wash. The work on this project was completed in July, 1908.

He was next employed as Construction Engineer by the United States Reclamation Service, Yakima Project, North Yakima, Wash., from July, 1908, until March, 1909, at which date he resigned to become City Engineer of Vancouver, which is situated across the Columbia River from Portland, Ore., and is the seat of a prominent United States Military Post. His work here consisted of general municipal improvements on streets, parks, and sewers. In addition to his duties as City Engineer, Mr. Lotter was Engineer for the Washington-Oregon Corporation for water, gas, and electric railway systems. In this capacity he built the electric road from Vancouver to Sifton, Wash. He also served during this period, from March, 1909, until January, 1912, as Engineer for the Town of Camas, Wash.

From January, 1912, until July, 1913, he was engaged in general engineering practice and contracting at Vancouver, and was also City Engineer for Kalama, Wash., directing street and sewer improvements.

On leaving the Pacific Coast in July, 1913, Mr. Lotter became associated with the United States Bureau of Public Roads, in the Department of Agriculture, in the capacity of Senior Highway Engineer, with headquarters in Washington, D. C.

During the next seventeen-year period until the time of his death (1913-1930), the territory over which he worked included large sections of the eastern half of the United States, ranging from the Gulf Coast States and the Southeastern Atlantic Seaboard States to as far north as Michigan and then back to the States of the Middle West.

Mr. Lotter's first work with the Bureau of Public Roads consisted of the construction of post roads in the State of Mississippi, followed by similar work in the State of Iowa. The peace-time road program was interrupted in 1917 by the entrance of the United States into the World War, and, in consequence, a large part of the Bureau's activities was directed toward war work, in the construction of roads to and through cantonments to facilitate the mobilization of troops. Mr. Lotter was employed in this work at Camp Dodge, Des Moines, Iowa, and on its completion early in 1918, he was transferred to Montgomery, Ala. He traveled in the interests of the Bureau through the States of Alabama, Georgia, Florida, and South Carolina, and assisted in the construction of roads to Camp Wadsworth, South Carolina, and in the road through Camp Sevier, South Carolina.

In July, 1918, Mr. Lotter was transferred to Federal District 5, with headquarters at Omaha, Nebr., where he remained until July, 1919. While in this District, he also assisted in the road building at Fort Riley, Kansas.

During this period of a year, the war having ended, work in the Bureau was intensively re-directed to the general road-building program. Mr. Lotter traveled a large part of the time, inspecting numerous Federal Aid road projects in the States of Iowa, Nebraska, Missouri, and Kansas, the four States that comprise Federal District 5.

In July, 1919, it was decided by Bureau officials in Washington to establish a Bureau Office for Federal road work in the State of Missouri, with headquarters at Jefferson City. Mr. Lotter was selected to become the head of this responsible post. During the eleven years that followed, he worked tirelessly in every way for the improvement and expansion of the road system of his native State. He traveled throughout all Missouri, giving advice and making inspections on hundreds of Federal Aid road projects. He had been in failing health for several years, but kept up his work until he was seized with the sudden attack of heart failure which caused his death at his home in Jefferson City, on September 9, 1930.

Ever an ardent and enthusiastic engineer, Mr. Lotter, at all times, exemplified and advocated the high standards of honor, diligent work, and efficiency for which his profession stands. In every community in which he lived he was regarded as a useful and highly respected citizen, and he was held in high esteem by his professional associates everywhere. His fine character, interesting personality, and ready wit made for him a nation-wide circle of friends.

Mr. Lotter served as a Colonel on Governor Baker's personal staff. He took an active interest in community affairs at Jefferson City, serving for a time as Secretary of the Commercial Club. He was a member of the Jefferson City Rotary Club, Jefferson Lodge, No. 43, A. F. and A. M., Jefferson City Chapter No. 34, Royal Arch Masons, Ezra Council No. 32, R. and S. M., and Prince of Peace Commandery No. 29, Knights Templars. He served as a member of the Advisory Council of the Order of DeMolay. He was a member of the Knights of Pythias Lodge. He was also a loyal member of the Presbyterian Church at Jefferson City and served as a Trustee, an Elder, and as Assistant Sunday School Superintendent.

He was known and loved for his interest in young people. He took great joy in the church, fraternal, school, and athletic activities in which young people engage, giving much of his time and attention to these enterprises.

Mr. Lotter's character has been well epitomized in the following editorial from the *Jefferson City Capital News:*

"'The evil that men do lives after them; the good is oft interred with their bones.' Thus spake Shakespeare, but like many other sayings of the Bard of Avon, it is not true to the experience of the race. The good men do lives after them much longer than does the evil, because a good influence is more lasting than an evil one.

"Take the case of Howell Lotter. He was a good man; his hands were full of kind deeds; men, women, and children loved him; he so lived to set in motion a wave of influence for the good that will widen as it runs, as the years come and go. How true it is of Howell Lotter that he still lives, though dead.

"At the Rotary luncheon Monday, strong men spoke with emotion of his admirable qualities of head and heart. A tribute to his ability and fidelity was paid by Chief Engineer Cutler. Others spoke of him as a Churchman, a friend, a neighbor, a Rotarian, a public-spirited citizen, and as a Christian gentleman. All paid him the highest encomium of love and respect.

"Cut down in the midst of a useful career, his days were all too few. But he left his family and his friends and comrades the rich heritage of a good life. The world is better for Howell Lotter having lived in it. The Memorial Service at the Rotary luncheon was a beautiful tribute to a life that enriched this community."

On June 18, 1902, Mr. Lotter was married at Jefferson City to Gertrude C. Geisberg, who survives him, with two daughters, Dorothy and Charlotte.

Mr. Lotter was elected a Member of the American Society of Civil Engineers on September 9, 1919.

JACOB LOTT LUDLOW, M. Am. Soc. C. E.[1]

Died August 18, 1930

Jacob Lott Ludlow was born at Spring Lake, N. J., on December 20, 1862, the son of Samuel and Nancy (Johnson) Ludlow. He obtained his Civil Engineering Degree at Lafayette College at Easton, Pa., in 1885, and five years later was awarded the Honorary Degree of Master of Science by the same College.

Soon after his graduation Mr. Ludlow located at Winston, N. C., at that time a sister city to the old Moravian Town of Salem and, later, as President of the Board of Trade of Winston, he led the successful campaign for the consolidation of Winston and Salem into one municipality now known as Winston-Salem, N. C.

He was the first City Engineer of Winston and as such succeeded not only in establishing a sanitary sewerage system in that city, but materially assisted in the establishment of one of the first street railway systems to be installed in the United States. He started a program of paving which to-day has made Winston-Salem one of the most thoroughly paved cities in the South.

During the boom years following 1890, Mr. Ludlow did much sub-division work throughout the South and, to-day, many a city can thank him for its splendid parks which he with far-sighted vision invariably included as a part of his plan, dedicating such areas forever to public use.

However, the greater part of his active life was spent in the development of public water supplies throughout the South. For a period of twenty-five years, he was the Engineer Member of the North Carolina State Board of Health and untiringly directed his efforts toward improving sanitary conditions in the State, drafting the laws for the protection of public water supplies that have placed North Carolina well in the forefront in such matters. He was the only lay member of the North Carolina Medical Society.

As a practicing Municipal Engineer there are few cities and towns in the South Atlantic States that were not served by him in some capacity during his long career.

For eight years he served as Chief Engineer of the North Carolina National Guard with the rank of Colonel. This was during the period prior to the World War when the National Guard of the country constituted an important part of the National defense. During this time he designed and established the camp equipment and rifle range at Camp Glenn, near Morehead City, N. C.

---

[1] Memoir prepared by William M. Platt, M. Am. Soc. C. E.

During the World War, Colonel Ludlow supervised the construction of Camp Greene, at Charlotte, N. C. Following this, until the close of the war, he was Supervising Sanitary Engineer for the United States Shipping Board at the many shipyards in the South Atlantic and Gulf States, paying particular attention to the correction of the drainage in the neighborhood of the yards and the removal of the malarial conditions that existed there.

Following the World War Colonel Ludlow confined himself to a consulting practice and also acted as Consulting Engineer for the Ludlow Engineers, of which organization Major E. W. Myers was the active head. After the death of Major Myers, Colonel Ludlow continued the organization, serving as active head to the time of his death. The organization has been discontinued.

During this period, in 1924 and 1925, Colonel Ludlow also served as a Member of the Engineering Board of Review of the Sanitary District of Chicago on the lake-lowering controversy.

Colonel Ludlow was a member of the Phi Delta Theta Fraternity, a York Rite Mason, a Shriner, a member of the American Institute of Consulting Engineers, the American Public Health Association, the American Water Works Association, and the North Carolina Society of Engineers. He was also a member of the Rotary, Twin City, and Cosmos Clubs of Winston-Salem, and a member of the Presbyterian Church.

During his lifetime he held many prominent positions not previously enumerated, among which may be included that of Chairman of the Sanitary Engineering Section of the American Public Health Association, President of the North Carolina Section of the American Water Works Association, Director of the American Association for the Study and Prevention of Tuberculosis, Director of the North Carolina Tuberculosis Association, National Councillor of the Chamber of Commerce of the United States, and Past Potentate of Oasis Temple, A. A. O. N. M. S.

Colonel Ludlow was married in 1887, to Myra Margarette Hunt, of Easton, and to them were born three daughters, Mrs. Joseph F. Cannon, of Concord, N. C., and Mrs. Henry B. Shelton and Mrs. R. A. McBrayer, of Winston-Salem, who, with Mrs. Ludlow, survive him.

The widespread editorial comment throughout the South and particularly throughout North Carolina where he was more intimately known, is concrete evidence that in the passing of Colonel Jacob Lott Ludlow the South lost one of its great benefactors.

Coming there as a young man soon after his graduation from college, to a field where municipal engineering was practically unknown and which had scarcely recovered from the ravages of the Civil War, he early became a large factor in the upbuilding of the South, and his many activities during the past forty-five years, aside from his engineering work, stamped him as no less a great citizen than a great engineer.

In the sanitary engineering field Colonel Ludlow was a pioneer and with the construction of the separate sanitary sewers in the City of Raleigh, N. C., in 1888, one of the first systems of that type in the world, was begun the development of a technique that has become standard practice throughout the Southern States.

As an ethical practitioner he was without a peer, never failing in the conduct of his work to recognize his position as arbitrator and the rights of the contractor as well as the client. The profession as well as the Society owes much to Colonel Ludlow for the example set for the guidance of the younger engineer.

These are some of the high lights of an active, unselfish career of forty-five years, which only a lengthy biography could fully cover and which at every step reflected credit upon the Engineering Profession.

Colonel Ludlow was elected a Member of the American Society of Civil Engineers on March 3, 1897.

JOHN LUNDIE, M. Am. Soc. C. E.[1]

DIED FEBRUARY 9, 1931

John Lundie was born in Arbroath, Scotland, on December 14, 1857, the son of James and Anne Honeyman Lundie. On his father's side, he was descended from the Lundies of Forfarshire, in a direct line from Sir John Lundie who was keeper of Stirling Castle in the Fifteenth Century. His mother's ancestry derived directly from James the Fifth, the Stuart King, one of whose descendants married Andrew Honeyman, Bishop of Orkney, in the Seventeenth Century.

After his graduation from the High School of Dundee, Scotland, in 1873, Mr. Lundie served a pupilage of four years in the office of the Harbor Engineer of the Port of Dundee, where he obtained some excellent experience in concrete sea walls, land reclamation, locks, graving docks, pumping machinery, harbor-railway work, power machinery, surveying, and general hydrographic engineering. He later entered the University of Edinburgh where he finished the four-year course in three years and was graduated in 1880 with the degree of Bachelor of Science in the Department of Engineering. He was First Prize man in Mathematical Physics.

Immediately after his graduation Mr. Lundie came to the United States and started on railroad work in Oregon and Washington Territory, continuing thereat for four years. During this time he was occupied with mountain preliminary and location surveying, general construction, maintenance, and yard work, besides having complete charge of the building of the Table Rock Tunnel.

From 1884 to 1885 he was engaged in private practice in Chicago, Ill., devoting his attention mainly to municipal work. It was during this time that he became a citizen of the United States.

---

[1] Memoir prepared by J. A. L. Waddell, M. Am. Soc. C. E., and Edward J. Murphy, Esq.

In 1886 Mr. Lundie entered the employ of the City of Chicago, as Assistant Engineer in charge of work, and continued in this position during four years under the direction of Francis Murphy, Harbor Engineer. He obtained experience in sewerage, steel work, water supply, tunneling, and bridges. He made the preliminary survey for the Chicago Drainage Canal and designed several of the city's bridges. For the last eighteen months of this service he was Principal Assistant City Engineer. One of his colleagues at this time was the late John E. Ericson, M. Am. Soc. C. E., who died in 1927, after serving as City Engineer of Chicago for nearly forty years.

In 1890, Mr. Lundie became Engineer and General Agent in Chicago for the King Bridge Company, of Cleveland, Ohio. During the four years that he held the position he designed and erected numerous bridges and other structures, and contracted for and erected the structural steelwork of four of the World's Fair Buildings and the Illinois Central Railroad Train-Shed at Chicago.

He then returned to private practice in Chicago, designing for that City its first low-level drainage system, as well as the sewerage system of Kenosha, Wis. He did his first electrial engineering work in 1895 for the Chicago and Worth Electric Railway Company. Also, he investigated and reported on an artesian water supply system for the City of Memphis, Tenn. During this investigation Mr. Lundie developed a method for the determination of the yield of artesian water areas.

In 1896, at the request of the late C. P. Huntington, F. Am. Soc. C. E., he visited the Isthmus of Panama, in order to make a survey and examination of the Pacific Mail Company's property at Colon and report thereon.

One of Mr. Lundie's principal achievements belongs to this period of his career, namely, an investigation and report on the application of electricity to the suburban train service of the Illinois Central Railroad. He spent an entire year in studying the subject; and during the research he enunciated the principle of "rapid acceleration", indicating the advisability of utilizing a high percentage of the weight on the driving wheels in economic movement of certain railway operations. From the results of this research he prepared a thesis containing a record of his original investigations, presented it to his Alma Mater, the University of Edinburgh, and received therefor the degree of Doctor of Science. This thesis enunciated the principle and economy of rapid acceleration, the scientific power rating of electric railway motors, and the law of fluctuations in power stations, and also established the Lundie formula for train resistance. He made some similar tests of train resistance on the South Side Elevated Railroad of Chicago, and some automobile tests for the *Chicago Times-Herald* competition.

In 1898, he was called to New York, N. Y., by the Sprague Electric Company to solve some heavy traction problems; and soon afterward he engaged in private practice in New York City, where he made many electric motor power tests for the Brooklyn Elevated Railroad, the Boston Elevated Railroad, and the Brooklyn Rapid Transit Companies.

About this time, Dr. Lundie reported on the power handling of freight at the Savannah Terminal of the Georgia Central Railroad Company, and, in the course of this investigation, he designed the first combined electric hoist and traveler, since called a "telpher". He designed and patented also the Lundie Ventilated Rheostat, now used quite extensively.

Dr. Lundie was widely consulted by various railways and other companies in the East, West, and South, on important electrification problems, and was called to London, England, on the electrification of the Metropolitan Underground System of Railways. He was retained thereon for two years, after which Sir William Preece, of London, sent him to Canada on some important electric railway work.

In 1904, he reported to the General Electric Company on the conditions and prospects for water power development and the utilization of electric power on the Isthmus of Panama; and while he was there he was retained by the Panama-American Corporation to direct its affairs on the Isthmus. After that, he did considerable important electrical engineering work in the States of Alabama, Tennessee, and New York.

Dr. Lundie served as efficiency expert for the Tennessee Coal, Iron, and Railroad Company, introducing much greater efficiency in railroad operation, especially by applying, with remarkable success, the "straw boss" system among the negro laborers.

In 1913, he designed and patented the Lundie Tie-Plate, an appliance for the promotion of safety, economy, and ease of movement on railways, and, later, a duplex rail-anchor, and thereafter he devoted a large portion of his time and energy to the exploiting of these inventions. The work thus involved became eventually so extensive and onerous that there was formed the Lundie Engineering Corporation to take care of it, Dr. Lundie being its President and principal stockholder until the time of his death. This shifting of the burden of commercial interests to other shoulders left Dr. Lundie time to devote himself mainly to research and technical work.

In 1921, he became associated with the United Central America Corporation as its Technical Adviser in connection with its engineering projects.

Dr. Lundie was a member of many professional scientific organizations, among these being the American Institute of Consulting Engineers, the American Association for the Advancement of Science, the American Iron and Steel Institute, the American Institute of Electrical Engineers, and the American Railway Engineering Association.

He was also a Mason and a Knight Templar, and a Fellow of the Royal Society of Arts, of London.

He was a member of the New York Athletic Club of more than thirty years standing; of the Old Colony Club, the Railroad Club, and of many other societies, such as the American Iona Society, St. Andrew's Society, etc.

In 1906, Dr. Lundie was married to Iona Oakley Gorham, a grandniece of former Governor Sterling Price of Missouri. She died in 1925. In 1929, Dr.

Lundie was married to Mrs. Alice Eddy Snowden, the widow of Dr. Albert A. Snowden. In addition to Mrs. Lundie, he is survived by two brothers, Mr. Robert Honeyman Lundie, a publisher of Dundee, Scotland, and Mr. William Honeyman Lundie, of Chicago; and two sisters, Mrs. George Davidson and Miss Georgina Lundie, both of Adelaide, South Australia.

Although in delicate health for some time, Dr. Lundie had remained active in business until a brief illness from pneumonia caused his death on February 9, 1931. His ashes are buried in Oakland Cemetery, Dallas, Tex., beside those of his first wife.

He will be missed not only in the scientific world to which he contributed such brilliant and valuable services, but by a host of associates who knew him as a cultured gentleman and as a generous, high-minded friend.

Dr. Lundie was elected a Member of the American Society of Civil Engineers on July 4, 1888.

HENRY MARTYN MacKAY, M. Am. Soc. C. E.[1]

Died October 25, 1930

Henry Martyn MacKay was born at Plainfield, Pictou County, Nova Scotia, on June 18, 1868. His father, Angus MacKay, had come to Canada in 1821 from Sutherlandshire, Scotland, and married Elizabeth MacKay, a native-born Nova Scotian, although no relation to his own branch of the family.

Angus MacKay was a staunch supporter of the Established Church of Scotland, and active in promoting the educational interests of the country community in which he lived. It was to be expected that his own sons and daughters should receive the best schooling his limited means could afford. Therefore, when he had absorbed all that the local teachers could give him, young Henry was sent to Pictou Academy, an institution which has an extended and excellent reputation.

From Pictou Academy, Henry MacKay entered Dalhousie University in Halifax, Nova Scotia, from which he was graduated in 1888 with the degree of Bachelor of Arts, and with high honors in mathematics and mathematical physics. Although his University career convinced him that his natural aptitudes were scientific rather than classical, he was not sure that they were sufficiently strong to warrant his adoption of engineering as a life work; his personal tastes, however, inclined in this direction. His native shrewdness and caution impelled him to consider the matter of his future carefully; if he were to follow the Engineering Profession, he must needs have more specialized education, and for that money would be required. He, therefore, spent the three or four years following his graduation from Dalhousie in teaching, and then entered McGill University, Montreal, Que., Canada, from which he received the degree of Bachelor of Science in Civil Engineering in 1894, winning the Governor-General's Medal for the highest standing in his class in any course.

---

[1] Memoir prepared by R. DeL. French, M. Am. Soc. C. E.

Immediately after his graduation from McGill University, Mr. MacKay entered the service of the Survey of Tides and Currents of Canadian Waters, under the late Dr. W. Bell Dawson. Here he remained until 1897, when his old school, Pictou Academy, called him to be Mathematical Master, which position he occupied until 1899. He was eminently successful as a teacher.

Henry MacKay early developed an intense interest in structural engineering. When, therefore, the opportunity arose of entering the employ of Waddell and Hedrick, Consulting Engineers, of Kansas City, Mo., he accepted it instanter. At that time, this firm was well-known in America, and its reputation was second to none. In addition, the Senior Member was a graduate of McGill University and a personal acquaintance of Mr. MacKay.

From 1899 to 1904, he served this firm in various capacities. For a few months, at the beginning of his term of employment, he was engaged in drafting and design in its Kansas City Office, but in November, 1899, he was appointed Resident Engineer on the construction of the Iowa Central Railroad Bridge, at Eddyville, Iowa, and thereafter was always in the field. From April to November, 1900, he was in charge of the erection of the Agnes Avenue Viaduct for the Metropolitan Street Railway Company, of Kansas City.

In November, 1900, his firm sent him to Mexico on the bridge work of the Vera Cruz and Pacific Railway, for which its members were acting as Consulting Engineers. For about a year Mr. MacKay was stationed at the Pahaloapam Bridge, and, finally, in October, 1902, he was given charge of all permanent bridges on the line, about one hundred in number and varying in span from 20 to 225 ft.

In June, 1903, he returned to the United States and served, successively, as Resident Engineer on the James Street Bridge and on the reconstruction of the Ohio Avenue Bridge, both of which are in Kansas City.

Dr. H. T. Bovey, then Dean of the Faculty of Applied Science at McGill University, had never lost sight of his brilliant graduate, and repeatedly had tried to induce him to return to his Alma Mater as an Instructor; finally, in December, 1904, Mr. MacKay accepted the appointment of Special Lecturer in Civil Engineering. A natural-born teacher, in love with his work, he was rapidly promoted through the grades of Assistant Professor and Associate Professor to William Scott Professor of Civil Engineering and Head of the Department, in May, 1908.

Always a strong proponent of thorough instruction in the fundamentals of engineering, Professor MacKay devoted the next sixteen years to the building up of his Department. Firm in his convictions, but fair in his judgments, he earned the respect and admiration of his colleagues to such an extent, that when the position of Dean of the Faculty became vacant through the retirement of Dr. Frank Dawson Adams, he was the logical choice of the Corporation of the University for the office, to which he was appointed in June, 1924.

Although Dean MacKay's innate modesty and the fact that his major interests lay in the field of teaching and collegiate administration tended to throttle his output of professional work, he yet found time to contribute materially to the progress of his especial branch of civil engineering, namely, bridges and structures. All the time that his strict sense of duty to the University would permit him to use, was occupied by consulting engagements.

Perhaps the most noteworthy of these was his investigation of the behavior of full-sized steel columns under heavy loads, for the St. Lawrence Bridge Company, Limited, of Montreal, in connection with the construction of the Quebec Bridge, and described in the Appendices of the report of the Board of Engineers on that structure. Another investigation which led to important results was that on the effect of haunching I-beams with concrete, which he conducted jointly with the late Professor Peter Gillespie, of the University of Toronto, and with Professor S. Baulne, of L'Ecole Polytechnique, in Montreal. His last consulting engagement was as a member of the Advisory Board of Engineers of the Montreal Harbor Bridge over the St. Lawrence, which was opened in May, 1930.

In recognition of Dean MacKay's efforts in the field of engineering education and of his professional attainments, Dalhousie University, in 1929, conferred on him the degree of Doctor of Laws, *honoris causa.*

Dean MacKay possessed the rare faculty of making a complicated matter appear clear, simple, and obvious. To this talent was perhaps due his success as a teacher. Singularly gentle in personality, he could be inflexible when occasion demanded, although always inspiring the respect and love of his colleagues and the hundreds of students who passed through his hands in the twenty-six years during which he had served the University. Although abhorring enforced or artificial intimacy, such was his character that he was the repository of the confidences of staff and students alike.

In 1910, he was married to Lilian Norton Evans, who, with two children, Ian Norton and Elizabeth Nora, survives him.

Although essentially a home-lover and happiest in the heart of his family, Dean MacKay was ever ready to support any worthy work both financially and by means of his own services. He was a member of the Church of the Messiah (Unitarian), and of the University Lodge, A. F. and A. M.

He was affiliated with the following professional societies: the Engineering Institute of Canada, which he served as Counsellor; the American Society for the Advancement of Science; the Society for the Promotion of Engineering Education; the American Society for Testing Materials; the Canadian Engineering Standards Association; the Canadian Railway Club; and the Society of the Sigma Xi. For many years he had been a member of the Honorary Advisory Council for Scientific and Industrial Research, a Federal Government body. He was also a member of the University Club, the Faculty Club, and the Art Association of Montreal.

Dean MacKay was elected a Member of the American Society of Civil Engineers on July 10, 1907.

WILLIAM WALTER MACLAY, M. Am. Soc. C. E.[1]

DIED JUNE 8, 1929

William Walter Maclay, the son of Archibald and Julia Ann (Walker) Maclay, was born in New York, N. Y., on March 27, 1846. He was a descendant of Henry Slater, one of the most notable pioneers of Colonial days who came from England in 1709 and settled for a time in Virginia. Henry Slater subsequently purchased a tract of land and made his home on what was then known as "Chestnut Ridge", about nine miles northwest of Baltimore, Md. He became prosperous as a planter and in 1742 erected the first Baptist Church in Maryland. It is one of the famous landmarks in American history and was built contemporaneously with Faneuil Hall, Boston, Mass., the old Lutheran Meeting-House, Philadelphia, Pa., and the Baptist Church, Ringwood, N. J.

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

Mr. Maclay was a maternal grandson of Isaac Walker (1786-1868), an original member of the Association of Defenders of Baltimore in 1814, a private in Captain Thomas Watson's Company, 39th Regiment, Maryland Infantry, and among those who served with Lt.-Col. Benjamin Fowler at the Battle of North Point.

Archibald Maclay, the father of William Walter Maclay, was a prominent physician of New York City (1812-92). His grandfather, the Rev. Archibald Maclay, was a graduate of the University of Edinburgh, Scotland, Pastor of the Mulberry Street Baptist Church, New York, and, later, served as Traveling Agent of the American and Foreign Bible Society. He was an able preacher and an earnest advocate of the Bible cause, and to this day his influence is felt among the English and Welsh-speaking people of the United Kingdom as well as the United States.

William Walter Maclay received his early education at the Grammar School of the University of the City of New York, and, in 1860, entered the United States Naval Academy, at Annapolis, Md., as a Midshipman. He was graduated in 1863, the youngest member of his class, and commissioned an Ensign. This was the class from which were chosen the Captains for most of the battle-ships in the Spanish-American War.

During the Civil War, in 1864-65, he served on the *Ticonderoga* in both bombardments of Fort Fisher and made a voyage around the world as Navigating Officer to Commodore Goldsborough on the U. S. S. *Shenandoah* (1865-69). During this period his letters in description of the places he visited, were contributed to various New York publications.

On the conclusion of his journey in 1866, Ensign Maclay was appointed by the Japanese Government on a Lighthouse Board to consider the lighting of Yeddo Bay. In the same year he was commissioned Lieutenant and, in 1868, Lieutenant Commander. He was subsequently appointed Fleet Lieutenant and Acting Fleet Captain of the United States Naval Asiatic Squadron. In 1869, he became Assistant Professor of Mathematics at the Naval Academy, Annapolis, which position he resigned in 1871.

He then pursued a course of study in Civil Engineering at the University of the City of New York and was graduated with the degree of Civil Engineer in 1873, having received the degree of Master of Arts in 1866 from the same institution. He was further honored by receiving the Norman Gold Medal from the American Society of Civil Engineers in 1878, for his paper entitled "Notes and Experiments on the Use and Testing of Portland Cement."[2]

---

[2] *Transactions*, Am. Soc. C. E., Vol. VI (1877), p. 311.

For many years Mr. Maclay served as Principal Assistant Engineer in the Department of Docks, New York City, on the construction of the new pier and bulkhead system. He was also prominently identified with the Portland cement industry of the United States. As a Civil Engineer using large quantities of this material and as a Portland Cement Expert and Manufacturer, he was one of the foremost men in bringing about the present development of the industry. It was upon Mr. Maclay's recommendation to the late James B. Eads, M. Am. Soc. C. E., that American Portland cement was used in building the jetties at the mouth of the Mississippi River. This occurred at a time when the use of such cement in the United States was in its infancy, as it had been made only in a small way by one company. Inasmuch as it was also the first public recognition of the ability of American manufacturers in this line, the subsequent expansion of the trade was accordingly greatly influenced.

In 1893, Mr. Maclay removed to Glens Falls, N. Y., where he built the large Portland Cement Works of which he was President and General Manager until about 1904, at which time he resigned. He then spent a few years in foreign travel, after which he returned to the United States and rebuilt a country home in Lee, Mass.

In addition to his paper on "Notes and Experiments on the Use and Testing of Portland Cement" previously mentioned, Mr. Maclay was the author of "Portland Cement for Engineering Works". He was a member of the Institution of Civil Engineers, of London, England, the International Society for Testing Materials, and a Corresponding Member of the New York Historical Society. He was also a member of the University and Manhattan Clubs of New York, and of The United States Naval Academy Graduates Association.

He died at his home in Lee, on June 8, 1929, in his eighty-fourth year. He lived only a little more than a year after the death of his wife, Marian Bensel Maclay, to whom he was married, in New York, on September 16, 1874. His daughter, Mrs. Robert S. Linn, and a grand-daughter, Mrs. Frank E. Carlson, survive him, both of whom live in the Middle West. A son, Walter, died in 1887.

Mr. Maclay was elected a Member of the American Society of Civil Engineers on November 6, 1872.

GEORGE ARNOLD McCARTHY, M. Am. Soc. C. E.[1]

Died November 13, 1930

George Arnold McCarthy was born at Moncton, N. B., Canada, on December 19, 1871. Educated at Moncton, he was graduated with the degree of Bachelor of Science at McGill University, Montreal, Que., Canada, and obtained his preliminary engineering practice during a period of seven years in the office of the Chief Engineer of the Intercolonial Railway Company, at Moncton.

Having definitely chosen engineering as his profession, Mr. McCarthy obtained an appointment first with the Canadian Pacific Railway Company in the Lake Superior District, and, later, with the Algoma Central Railway Company. From 1901 to 1905 he was Principal Assistant Engineer for the Canadian Niagara Power Company, at Niagara Falls, Ont., Canada; from 1905 to 1909, Chief Engineer of the Temiskaming and Northern Ontario Railway Commission; from 1909 to 1912, Western Manager for Messrs. Smith, Kerry, and Chace at Calgary, Alberta, Canada, and with the same firm in Toronto, Ont., Canada, from 1913 to 1914, after carrying out some work as Resident Engineer on the Feather River Development for the Oro Development Company, of San Francisco, Calif.

---

[1] Memoir prepared by R. C. Harris, Esq., Toronto, Ont., Canada.

In 1914, Mr. McCarthy entered the service of the Department of Works of the City of Toronto, in which he remained until his death. As Engineer in charge of the Railway and Bridge Section, he was responsible for the administration of the civic car lines, and for the construction of several large works, including the Bloor Street Viaduct and the Strachan Avenue and Crawford Street Bridges. From 1922, he was in charge of the Sewer Section of the same Department, and many large works of sewer construction were carried out under his direction, notably, the North Toronto Sewerage System and Sewage Treatment Works, the largest complete system ever constructed by the City.

Mr. McCarthy was always active in general engineering matters, and spent much time in the service of his profession in the various societies with which he was connected. He was a member of the following: The Engineering Institute of Canada, the American Railway Engineering Association, the American Institute of Electrical Engineers, and also of the Association of Professional Engineers of the Province of Ontario. He served as a member of Council of the Engineering Institute of Canada during 1917, 1918, and 1919, and as Chairman of the Toronto Branch for one year. In engineering circles, he will long be remembered as a man whose presence was a help and an inspiration to his fellows.

He was married in Montreal on October 21, 1901, to Jennie M. Moffatt. Besides his widow, there are three children surviving, namely, Doris McCarthy, of Toronto, Douglas F. McCarthy, of Kapuskasing, Ont., Canada, and Dr. K. C. McCarthy, of Toledo, Ohio.

Mr. McCarthy was elected a Member of the American Society of Civil Engineers on November 6, 1907.

WILLIAM EDWARD McCLINTOCK, M. Am. Soc. C. E.[1]

Died March 2, 1930

William Edward McClintock was born in Hallowell, Me., on July 29, 1848, the son of Captain John McClintock, a well known navigator, familiar with every sea, who crossed the Pacific with an ordinary watch for a chronometer and a school atlas for a chart, and Mary Bailey (Shaw) McClintock. On his paternal side he was of Scotch-Irish ancestry, descended from William McClintock, a defender in the Siege of Londonderry in 1689. His earliest American ancestor was William McClintock, who came to the United States, a babe in arms and whose parents settled in Medford, Mass., in 1735. On his mother's side, he was descended from the Rev. John Bailey, a prominent Puritan clergyman.

---

[1] Memoir prepared from data on file at Society Headquarters.

Mr. McClintock's early education was acquired in the public schools, supplemented by a four-year course at Hallowell Academy, and one year at Kent's Hill Seminary, at Readfield, Me. His early manifested taste for engineering was inherited from his paternal family, his grandfather, William McClintock, having been an expert land surveyor, some fine examples of whose work are now on file in the State of Maine.

On the completion of his preparatory education Mr. McClintock entered a period of training for his profession, including both office and field work, and he also received instruction from a private tutor. His first work as an engineer was in connection with the United States Coast and Geodetic Survey with which he was engaged from 1867 to 1876 on work in Maine, Massachusetts, New York, North Carolina, South Carolina, Georgia, Florida, Louisiana, and Mississippi. From 1876 to 1879 he was employed on a survey of the City of Portland, Me., and from 1877 to 1879 on a survey of Boston Harbor, and also on a re-location survey for the Boston and Maine Railroad Company, including all its branches in Massachusetts. In 1880, he was elected City Engineer of Chelsea, Mass., which position he retained until 1890 when he went into business with Mr. J. Leslie Woodfall under the name of McClintock and Woodfall.

Mr. McClintock's special engineering works have included surveys for the South Pass Jetties, at the mouth of the Mississippi River; surveys for the improvement of the harbors of New York, N. Y., Boston, Mass., and Portland, Me.; surveys for the improvement of the Saco, Savannah, Pamlico, St. Mary, St. John, and Nassau Rivers; and the installation of sewerage systems for the cities and towns of Chelsea, Revere, Gardner, Westfield, East Hampton, Andover, Lenox, Natick, and Lexington, Mass.; Bennington, Vt.; Bath and Calais, Me.; Exeter, N. H.; and St. Stephen and Milltown, N. B., Canada. He also served as Consulting Engineer on sewer and water-works construction in Holyoke, Spencer, North Brookfield, North Attleboro, and several other towns in Massachusetts as well as in other States.

Mr. McClintock was actively identified with the Good Roads Movement in Massachusetts and, in its advocacy, he wrote various papers, and delivered addresses in nearly every city and town in the Commonwealth. In 1892 the old League of American Wheelmen, which had been agitating the question of better roads for several years, was publishing a magazine in an effort to stimulate public interest. At that time Mr. McClintock was building roads in various towns in Massachusetts, Maine, and New Hampshire. Through the efforts of the League, Governor W. E. Russell appointed a Commission to investigate and report on the improvement of highways in Massachusetts, and Mr. McClintock was appointed as the Engineer Member, together with George A. Perkins, a lawyer, and Nathaniel Shaler, widely known as Professor of

Geology at Harvard University. The result of the studies of this Commission showed that outside the thickly settled communities there were practically no good roads in the State.

In 1893, the Legislature passed an Act providing for the establishment of the Massachusetts Highway Commission and these same three men were appointed to form the new Commission. Mr. McClintock was subsequently re-appointed by Governors Greenhalge, Walcott, Crane, Guild, and Bates. In 1898, he was made Chairman of the Commission which position he retained until 1908 when he relinquished this work to give his services to his own home town which had been devastated by fire. He may be identified as a pioneer in the Good Roads Movement, and had much to do with organizing the Engineering Staff that was to carry out the Massachusetts system of road-building, a system which has been widely studied and copied throughout the country. He was nationally known as the "Father of Good Roads."

It was during his administration as Chairman of the Massachusetts Highway Commission that the increasing motor-vehicle traffic showed the impracticability of the water-bound macadam highway which until that time had been considered the finest road in existence. Under Mr. McClintock's leadership the bituminous macadam highway was devised, and it has since been the standard for the superior motor highway, except in sections of the heaviest travel where concrete has been adopted.

In 1908, came the Chelsea fire, during which more than 17 000 people were rendered homeless, and churches, public buildings, and schools were destroyed. Mr. McClintock was the man to whom every one looked to take charge of the relief operations, and so effectively was this done that when it was decided that Chelsea was to have an emergency commission form of government for five years, Mr. McClintock was selected by Governor Eben Draper as Chairman of the Board of Control. In this capacity he again had an opportunity to show his executive and constructive ability in the rebuilding of the city, and to this end he gave unremittingly of his time and service. Despite some opposition and many difficulties, this Board brought Chelsea out of its stricken condition. It rebuilt all the public buildings, laid out new streets, and left the city with better structures than it had previous to the fire. It was while he was engaged whole-heartedly in this work that he was recognized by Governor Charles E. Hughes, of New York State, as the leading authority in the United States on the construction of modern highways, and he was invited to accept the Chairmanship of the New York Highway Commission. He refused this offer because he was devoted to his community and to the work on hand.

In 1913 when the city returned to the Mayor and Board of Aldermen form of government, he was appointed Superintendent of Chelsea Ferry which position he held until the ferry was discontinued. He then became connected with the State Department of Education, in its field of University Extension. He died at his home in Chelsea on March 2, 1930, in his eighty-second year.

Mr. McClintock served on the Board of Aldermen, and School Committee, and was a Vice-President of the Chelsea Savings Bank. He was one of the organizers and first President of the Massachusetts Highway Association, a member of the Boston Society of Civil Engineers, and, in addition, an Instructor in Highway Engineering at Lawrence Scientific School, at Harvard University. He was a member of the Robert Lash Lodge of Masons, and the Royal Arch Chapter of the Shekinah, and of the Review Club. He was prominently connected with the Church of the Redeemer (Universalist) of Chelsea.

He was married in Portland, Me., in 1873, to Mary Estella Currier. They had five children all of whom are living: William J., of Melrose, Mass., Francis B., Samuel, and Dorothy, of Chelsea, and Paul, of Hillside, N. J.

Mr. McClintock was elected a Member of the American Society of Civil Engineers on December 5, 1888.

LAURENCE FRANCIS McCOY, M. Am. Soc. C. E.[1]

DIED JANUARY 13, 1930

Laurence Francis McCoy was born on April 29, 1865, at Dumont, Iowa. He was the son of John and Anna (Coyle) McCoy, both of Irish extraction. His father was a Civil War veteran who engaged in farming after the war.

Mr. McCoy received his education in Iowa, having been graduated from Iowa State College in 1885. Following his graduation, he worked for three years on surveys in Illinois and Iowa. Becoming interested in the development of the Northwest, he went to Spokane, Wash., in 1888. Railroad location and construction were the phases of engineering he preferred, and his first position in the West was with the Seattle, Lake Shore, and Eastern Railway Company.

He worked on many of the D. C. Corbin lines, serving at various times as Locating and Chief Engineer. These railroads included: The Spokane Falls and Northern, in 1889 to 1891; the Nelson and Fort Shepherd, a Canadian continuation of the Spokane Falls and Northern, in 1893; the Columbia and Red Mountain, a line built in 1896 to facilitate the development of the Rossland mines; the Spokane International, in 1906; and the Flathead Valley, in 1910.

From 1891 to 1893 Mr. McCoy was employed by the San Francisco Bridge Company at Sandpoint, Idaho. During the time of slackened building construction in 1894 and 1895, he taught school in Whitman County, Washington.

From 1901 to 1904, he worked on several divisions of the Oregon Short Line Railroad. On the Boise, Nampa, and Owyhee Branch, he was in charge of both engineering and construction. In 1907, he was employed by the Idaho and Washington Northern Railroad Company; his work here continued for two years, during which time he served both as Locating and Chief Engineer.

Mr. McCoy went to Sudbury, Ont., Canada, in August, 1911, as Division Engineer for the Canadian Northern Railroad Company. The outstanding feature of this work was the great number of bridges and tunnels necessitated by the rough and irregular nature of the country. On the completion of the work in 1914, he returned to the West where he accepted the position of Chief Engineer for the Blackwell Lumber Company at Cœur d'Alene, Idaho, a position that he held until the time of his last illness. For this Company, which was committed to a policy of railroad construction to get out its timber, a series of switchbacks on the side of a steep mountain was Mr. McCoy's first project. He later located and constructed many branch lines to the Company's scattered holdings. Subsequently, his duties grew to include the letting of all contracts and the determining of amounts to be paid. In consequence his reputation for honesty and accuracy was well established.

---

[1] Memoir prepared by Edward G. Taber, M. Am. Soc. C. E.

Although his work was in varying locations, Mr. McCoy was greatly interested in public affairs. He seldom failed to vote and frequently wrote articles on public questions for the newspapers.

He was accepted by all who knew him as a man of unquestioned integrity and the highest sense of honor. He was happy and proud of the fact that he numbered as firm friends the men who worked for him, as well as those for whom he worked.

He was married on November 23, 1893, to Crissie Rusch, who, with two sons, Robert F. and Joseph P., and two daughters, Anna and Marie, survives him.

His private life was governed by the best of ideals, and his genial and sympathetic nature made him loved as well as respected by all those with whom he was associated.

Mr. McCoy was elected a Member of the American Society of Civil Engineers on December 5, 1911.

JOHN ALEXANDER McDONALD, M. Am. Soc. C. E.[1]

DIED JUNE 4, 1930

John Alexander McDonald was born in London, England, on January 10, 1856. As a young man he studied civil engineering for two years at King's College, London, and served as an articled pupil for three years to Messrs. Jessop and Appleby Brothers, of London and Leicester, England. He later held a position of responsibility in the Greenwich Works of this Company.

Subsequently, Mr. McDonald went to Australia and was appointed Assistant Engineer in the Roads, Bridges, and Sewerage Department, of the Public Works of New South Wales. Shortly thereafter he occupied the position of Chief Assistant Engineer and, later, that of Engineer in charge of the design and construction of all bridges. He was the sole patentee of "McDonald's Patent Expansion Rollers" for large bridges and received first awards in London, Chicago, Ill., Adelaide, South Australia, and Melbourne, Victoria, Australia.

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

Mr. McDonald was again employed in the Roads, Bridges, and Sewerage Department of the Public Works of New South Wales, this time as Principal Assistant Engineer. After fourteen years of service, he resigned and sailed for a six months' trip to America.

On his return, he was appointed Assistant Engineer in the Public Works Department of Perth, Western Australia. Later, he became Resident Engineer on the Freemantle Harbor Works and Executive Engineer in charge of this work. He was then engaged as Assistant Engineer-in-Chief for the Colony of Western Australia and for nine months he served as Deputy Chief Engineer in full charge during the absence of the Chief Engineer on a trip to England. The construction work under his control included new and extensive railway workshops, with plant and machinery; the Freemantle Harbor construction, on which 1 000 employees were engaged; the water supply and sewerage systems, including artesian bores, for Perth and Freemantle; the Coolgardie Water Supply System which proved to be one of the largest in the world; roads and bridges throughout the Colony; water supplies for the gold fields; as well as the construction of all railways and all harbor and river works in the Colony.

When Mr. McDonald had spent four and one-half years in this service, he resigned and went to South Africa, where he was engaged in mine surveying and assaying in Rhodesia, and in the designing of electric light stations in Cape Town, Cape Colony. Finally, he established a private consulting practice in Durban, Natal.

Following this period, he was appointed Assistant Engineer in the Town Engineer's Department of the Johannesburg Municipality. He then became Deputy Town Engineer and for an interval of six months served as Acting Town Engineer. He controlled road and bridge construction, as well as sewerage, water supply, and tramway construction. This work included a large ferro-concrete power house and running sheds, refuse destructors, storm-water drainage, quarries, parks, swimming baths, fire stations, and concrete traffic subways. All these developments were undertaken for a town 81 sq. miles in area.

On his resignation from this position after five years of service, Mr. McDonald returned to Western Australia where he became Engineer for water conservation works. In 1912, he acted as Engineer to the Gisborne Harbor Board and, in 1918, was Resident and Consulting Engineer to the Gisborne Borough Council which position he held for five years.

He was a member of the Institution of Civil Engineers and of the Institution of Mechanical Engineers of London. Mr. McDonald died in his seventy-fourth year at Gisborne, New Zealand. He had been in failing health since 1923 and succumbed to a serious attack of erysipelas.

He is survived by his widow, Blanche de Warrenne McDonald, to whom he was married on December 24, 1919.

Throughout his life, Mr. McDonald showed marked organizing ability and much kindly consideration for all employees under his control and was respected by all with whom he came in contact. His opinion was frequently sought when difficult problems were under discussion, and he was an expert witness in many cases relating to engineering matters. In many places, structures of unusual merit remain as a lasting monument to his memory.

Mr. McDonald was elected a Member of the American Society of Civil Engineers on February 5, 1890.

ANSON BURLINGAME McGREW, M. Am. Soc. C. E.[1]

Died February 26, 1931

Anson Burlingame McGrew was born at Scott Haven, Pa., on May 10, 1868. His early education was received in the public grade schools and the High School at West Newton, Pa., and at the Indiana State Normal School. He entered the Western University of Pennsylvania (now the University of Pittsburgh), Pittsburgh, Pa., in 1889, from which he was graduated in 1892, with the degree of Civil Engineer.

From 1892 to 1894 Mr. McGrew was engaged in private practice and was employed as Borough Engineer for various boroughs in the Pittsburgh District on municipal improvements. In 1894 he entered the Government Service, in the War Department, and, until 1918, was engaged as Assistant Engineer in the Cincinnati, Ohio, Wheeling, W. Va., and Pittsburgh Engineer Districts.

From 1918 to 1919, during and immediately following the World War Mr. McGrew was District Engineer in charge of the Nashville and Chattanooga, Tenn., Engineer Districts, in which capacity he supervised river improvement work on the Cumberland and Tennessee Rivers and their tributaries. In 1919, he returned to the Pittsburgh District and was employed first as Assistant and then as Associate Engineer in charge of the Navigation Division of the United States Engineer Office until his death, which occurred suddenly following a short illness at his home in Beaver, Pa., on February 26, 1931.

Mr. McGrew was a member of the Local Membership Committee, the Pittsburgh Section of the Society, a Registered Engineer in the State of Pennsylvania, a certified member of the American Association of Engineers, a member of the Society of American Military Engineers, the Engineers Society of Western Pennsylvania, and the Chambers of Commerce, of Pittsburgh and New Brighton, Pa. He was a member of the Federal Business Association, and, also, a Director in the Home Protective Savings and Loan Association, of New Brighton.

He was married, at Beaver, on January 1, 1895, to Jessie Galey, who, with one daughter, survives him.

In addition to his professional activities, Mr. McGrew found time to devote to public and community affairs. He was also untiring in his efforts to advance the Engineering Profession and to secure proper recognition for engineers. In his passing, his many friends and associates feel a distinct sense of loss.

Mr. McGrew was elected a Member of the American Society of Civil Engineers on June 4, 1902.

---

[1] Memoir prepared by Nicholls W. Bowden, M. Am. Soc. C. E.

CLARK ROGERS MANDIGO, M. Am. Soc. C. E.[1]

DIED MARCH 25, 1930

Clark Rogers Mandigo was born in Sherwood, Mich., on February 13, 1884, the only son of William R. and Ada Idella (Rogers) Mandigo. His early education was received in St. Paul., Minn., where he was graduated from Central High School in 1902, and in Fargo, N. Dak. In June, 1907, he was graduated from Harvard University with the degree of Master of Civil Engineering, having previously obtained a Bachelor of Arts degree from Harvard College in June, 1906.

Previous to his entrance into Harvard College, Mr. Mandigo had served for several months as Rodman in the Engineering Department of the Northern Pacific Railway Company, and was similarly employed during the summer months of 1904 and 1905 as Rodman and Inspector for the same Company.

After his graduation in 1907, Mr. Mandigo re-entered the service of the Northern Pacific Railway Company as Inspector and Superintendent of Construction on 30 miles of track rebuilding and grade revision from Kalama to Vancouver, Wash., in which position he remained until March, 1908. At this time he was made Civil Engineer and Superintendent of Construction in the Quartermaster Department at Large, U. S. Army, War Department, at Fort Logan, Colorado, in charge of the design and construction of buildings, sewage disposal, and water supply. Later, he was with the same Department at Fort Riley, Kansas, engaged in building roads, paving, water-works, and sewers.

From February, 1911, until October, 1916, Mr. Mandigo served as Assistant City Engineer (for a time as Acting City Engineer), of Kansas City, Mo. He was appointed to this position as the result of a civil service examination. He had engineering charge of street and sidewalk construction, repair work and grades, retaining walls, small viaducts, bridges, subways, municipal wharf and wharf house, water supply, and sewers, as well as office and routine administration. From October, 1916, to March, 1917, he was Advisory Engineer with the Portland Cement Association at Kansas City, for which he compiled promotional statistical data and lectures on the various uses of Portland cement and examinations of defective structures.

For several months prior to his entrance into the Army Mr. Mandigo was Consulting Engineer for the Western Paving Brick Manufacturers Association. He entered the First Officers' Training Camp in May, 1917, and was commissioned a Captain in the U. S. Army, assigned to the 314th Engineers, 89th Division, Camp Funston, Kansas. He saw overseas service with the American Expeditionary Forces in France. He was promoted to the rank of Major, Corps of Engineers, in September, 1918, and was honorably discharged on November 30, 1918.

From December, 1918, until April, 1923, Major Mandigo again served with the Western Paving Brick Manufacturers Association as Chief Engineer, in which capacity he had charge of all technical and experimental work in the manufacture of paving brick and in the design and construction of brick pavements. He wrote many technical papers on this subject and on the use of brick in construction work. He was recognized as a national authority generally on paving and road work. From April, 1923, to January, 1927, he was Secretary-Engineer with the Kansas-Missouri Contractors Association, and the Kansas Contractors Association, a branch of the Associated General Contractors Association.

---

[1] Memoir prepared by Webster L. Benham, M. Am. Soc. C. E.

In January, 1927, Major Mandigo was appointed Chief Engineer for the Oklahoma State Highway Commission. As such, he served with great credit to himself and the profession until April, 1929, when he resigned and accepted a position as Engineer with the Southern Surety Company of New York, with headquarters at Kansas City, which position he held until his death on March 25, 1930. He died at St. Luke's Hospital, in Kansas City, following a major operation.

Major Mandigo was a man of great ability, had a most pleasing personality, and was exceedingly popular with all with whom he came in contact. He was a Mason and a Shriner. He took a very active part in civic affairs and the Rotary Club in Kansas City, and was a Past-President of the Kansas City Engineers Club. He was active in the American Legion and was elected Grand Chef de Guerre of the "Forty and Eight", for the State of Missouri, a social organization of the American Legion. He belonged to several engineering societies, and was a member of the Kansas City Athletic Club.

He was married on December 31, 1907, to Gladys Irene Allen, of Worcester, Mass., who survives him. He leaves, also, two sons, James Allen and Robert Allen, and a daughter, Violet Barbara Allen, as well as his father and mother, Mr. and Mrs. William R. Mandigo, of St. Paul, Minn.

Major Mandigo was elected an Associate Member of the American Society of Civil Engineers on September 3, 1912, and a Member on January 20, 1920.

MARSDEN MANSON, M. Am. Soc. C. E.[1]

DIED FEBRUARY 21, 1931

Marsden Manson, the son of Robert E. and Sophia A. (Smith) Manson, was a native Virginian, having been born at Leewood, on February 14, 1850. Most of his active life, however, was spent in California to which State he journeyed in 1877.

Mr. Manson was graduated from the Virginia Military Institute in July, 1870, and obtained his first engineering experience as a Rodman on preliminary and location surveys for the railroad lines from Bristol, Tenn., to Cumberland Gap. He was an Instrumentman on surveys for the Atlantic, Mississippi, and Ohio Railroad, from Bristol to Norfolk, Va., and, again, on a survey for a narrow gauge railroad from Liberty, Va., to Rocky Mountain, N. C.

Following some municipal work at Lynchburg, Va., he became, in 1874, an Assistant in the Corps of Engineers, United States Army, on surveys for the extension of the James River and Kanawha Canal and on the making of plans for the locks of this canal from Chesapeake Bay to the Ohio River. The following year Mr. Manson served for some months as Assistant City Engineer of Lynchburg and, then, became Assistant Professor of Physics and Chemistry at the Virginia Military Institute, at the same time continuing his studies which brought him the degree of Civil Engineer.

---

[1] Memoir prepared by C. E. Grunsky, Past-President, Am. Soc. C. E., and Otto von Geldern and E. F. Haas, Members, Am. Soc. C. E.

On his arrival in California, he was, for a brief period, an Instructor at the Brewer Boys' School, at San Mateo, and then found employment to his liking in the newly created State Engineer Department of California. Under the State Engineer, William Hammond Hall, M. Am. Soc. C. E., it fell to Mr. Manson to make some special studies relating to the damage being done to the rivers and to farm lands by the debris from hydraulic mines. These studies included a determination of the amount of material carried in suspension by the Sacramento River and its tributaries—studies which could in large part be carried on by Mr. Manson at the University of California where he registered for and was granted the degree of Doctor of Philosophy.

In 1881, he again became, for a brief period, an Assistant in the U. S. Corps of Engineers, and was then made Chief Engineer of the California State Board of Harbor Commissioners which has jurisdiction over and operates the harbor and water-front facilities of San Francisco. After eleven years in this position, he was made a member of the San Francisco Sewer Commission which, however, had only a brief existence.

Private practice in San Francisco followed, which terminated, in 1896, with his appointment as a member of the State Highway Commission of California. Mr. Manson was made a member of San Francisco's first Board of Public Works in January, 1900. In connection with his experience on this Board, it is interesting to note that an unusual charter provision placed the responsibility for the condition of streets and sidewalks on the individual members of the Board and that, regardless of inadequate appropriations and the unreasonableness of such a provision, judgments were obtained against Mr. Manson and paid by him which, it is understood, in one year at least, exceeded his annual salary ($4 000).

During his three years in the City's service, from 1900 to 1903, Mr. Manson became deeply interested in San Francisco's endeavor to secure the right to store water on Tuolumne River and its tributaries. He resented the attempted abandonment of the Tuolumne River water supply project by the municipal authorities and at no small expense to himself spent much time at Washington, D. C., in the interest of the City of San Francisco, with the result that an adequate permit was granted to this City in 1908 by the then Secretary of the Interior, the Hon. James R. Garfield.

The later acceptance by the City of a modification of this permit, whereby the City made all her priority rights to river water subject to first call by the Turlock and Modesto Irrigation Districts and submitted to onerous requirements relating to water and power development, and to road building, was a great disappointment to Mr. Manson, upon whose recommendation the original bond issue for a Tuolumne River Project (the so-called Hetch Hetchy Project) had been authorized and voted.

Mr. Manson was made City Engineer of San Francisco in 1908 and served in this capacity for four years. Thereafter, he remained active in private professional practice to the end of his days. "Dr." Manson, as his friends called him, died in San Francisco, on February 21, 1931.

He was married to Samuella L. Chase, at San Francisco, on September 12, 1883, and three years after her death, which occurred in August, 1913, he married Julia D. E. Wright, daughter of the late G. Alexander Wright, Architect, of San Francisco, who, with her two children, Julius A. and Marsden W., mourn his passing.

Mr. Manson had a strong leaning to the study of scientific problems and was particularly interested in meteorology and in the study of the earth's pre-glacial climate. This brought him into contact with scientists the world over. His was a pleasant and charming personality. His fund of scientific information was poured out to every interested listener. One of his friends has written:

"Maintaining his mental activity to the end, he loved to dwell upon the things that might be, and undoubtedly will be, done to make this Coast, especially California, with its marvellous scenic and climatic offerings the playground of the nation."

Another friend writes:

"He rendered invaluable service to the State and few engineers have been held in such high esteem by his contemporary members of the profession."

Mr. Manson was elected a Member of the American Society of Civil Engineers on September 6, 1882.

CHARLES CYRIL MARTIN, M. Am. Soc. C. E.[1]

DIED JULY 11, 1903

Charles Cyril Martin was born on August 30, 1831, in Springfield, Pa., the son of James and Lydia (Bullock) Martin, and a descendant of John Martin who came from England about 1666 and settled in Swansea, Mass. On the distaff side he was descended from John Howland, a *Mayflower* passenger.

Mr. Martin's early years were spent on a farm in the wilderness of Northern Pennsylvania amid the difficulties and hardships of frontier life. He became familiar with the details of farming and lumbering before he was 20 years of age, and during this time also had availed himself of every opportunity for study and reading that came his way. Previous to his matriculation at Rensselaer Polytechnic Institute, at Troy, N. Y., he was an accepted local authority on land surveying and had taught in the local district school. Through private tutoring and teaching, he secured sufficient funds to maintain himself and complete his college course at Rensselaer from which he was graduated with high honors in 1856.

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

During this year, Mr. Martin remained at the Institute as a teacher, but in 1857 he declined a further professorship and accepted employment as Rodman on the Brooklyn, N. Y., Water-Works then in course of construction. A steady advancement secured for him the position of Assistant Engineer in charge of the construction of three of the reservoirs and eight miles of the great conduit.

In 1860, on the completion of the Brooklyn Water-Works, he was engaged as Draftsman for the Trenton Locomotive and Machine Manufacturing Company of New Jersey, of which he was later made Superintendent. In this position he had charge of iron bridge building, car and locomotive construction, etc.

At the outbreak of the Civil War, Mr. Martin was engaged on the construction of an iron bridge across the Savannah River, on the Savannah and Charleston Railroad. This work was necessarily abandoned and the iron which was intended for use in its construction was melted into shot and shell for the Confederate Armies. After many vicissitudes Mr. Martin reached home *via* Nashville, Tenn. On account of the war, the character of the work at the Trenton Shops was changed to include the manufacture of ordnance, of which part Mr. Martin served as Superintendent and General Manager. Part of his work consisted in making experiments on combinations of metal for heavy guns.

In 1864, he was employed by the United States Government as an expert in a series of boiler experiments at the Brooklyn Navy Yard. Soon thereafter he was appointed Superintendent of the laying of the four 48-in. water mains along Atlantic Avenue to the Ridgewood Reservoir, through which the water supply of Brooklyn has since been largely obtained. In the spring of 1865 he was made Chief Engineer of Prospect Park in Brooklyn and brought to a successful completion the work on the landscaping, lakes, bridges, drives, and drainage, including the great Park Well, which at that time was the largest well in the world.

In January, 1870, Mr. Martin entered the employ of the New York and Brooklyn Bridge Company as Principal Assistant Engineer and Superintendent under Col. W. A. Roebling. Mr. Martin's former experience in bridge construction and foundations proved of the utmost value in this work. From the inception of the New York and Brooklyn Bridge until its completion, he had full charge of the execution of the work, the employment of men, the purchase of materials, and the auditing of bills. In his nineteen years of service as Chief Engineer and Superintendent of this bridge, Mr. Martin was active, industrious, and competent in every way. In 1902, when the office was abolished by the consolidation of the Department of Bridges, he was appointed Consulting Engineer to the new Department, and 1903, the year of his death, marked as complete thirty-three years of activity on behalf of the Brooklyn Bridge and more than forty years of professional service on public works in Brooklyn, a city he had most ably and faithfully served.

Mr. Martin had tact and ability which, with his integrity, exercised a beneficent effect on all his subordinates and associates and made friends of every one with whom he came in contact. He performed a notable service in his frequent counsel and advice to young engineers and was able thereby to impart a self-confidence and encouragement to them seldom surpassed by any one man in his profession. Partly due to this fact he received the unanimous vote of the Alumni of the Rensselaer Polytechnic Institute for the head of that Institution—an honor he reluctantly declined.

He was married to Mary A. Read, the daughter of Brig.-Gen. Jonathan Read, of Rensselaer County, New York, on August 9, 1859. They had two sons, Charles B. and Kingsley L. Martin, and two daughters, Mrs. John J. Hopper and Mrs. George Blatchford.

Mr. Martin's death occurred from heart failure at the summer home of his son, Kingsley L. Martin, at Far Rockaway, Long Island, on July 11, 1903. In reporting his death to the Mayor, the Commissioner of Bridges said:

> "In the death, on Saturday, July 11th, 1903, of Mr. Charles Cyril Martin, Consulting Engineer of the Department of Bridges, the City of New York has sustained the loss of an eminent engineer, an able public officer, a wise counsellor, an honored citizen, and a man of unimpeachable integrity.
>
> "Identified with the public works of the City of Brooklyn for over forty years, he was intimately connected with the New York and Brooklyn Bridge from its conception. Entering its service on January 2d, 1870, as First Assistant Engineer under Colonel W. A. Roebling, he held that office until the Bridge was completed in 1883, when he was appointed Chief Engineer and Superintendent of the New York and Brooklyn Bridge. This office he held until January, 1902, when he was appointed Consulting Engineer to the Department of Bridges.
>
> "He served in each position with such faithfulness and success as to confer honor upon himself and dignity and distinction upon his profession."

Mr. Martin was elected a Member of the American Society of Civil Engineers on July 10, 1872, and served as Vice-President of the Society from 1894 to 1895. He was also an Honorary Member of the Brooklyn Engineers Club.

SHELDON ELTON MINOR, M. Am. Soc. C. E.[1]

DIED FEBRUARY 19, 1931

Sheldon Elton Minor was born in Northfield, Conn., on September 14, 1860, the son of Henry and Amelia (Heaton) Minor. His early life was spent in Litchfield, Conn., until the death of his father. The family then moved to Thomaston, Conn., where he attended High School. He afterward entered the Sheffield Scientific School of Yale University, New Haven, Conn., from which he was graduated with the degree of Bachelor of Philosophy in 1882, winning the prize in Civil Engineering.

Immediately after his graduation, Mr. Minor started his career as Rodman for a proposed South Pennsylvania Railroad, on which work he was engaged until July, 1883. He then went to Santiago, Cuba, from September, 1883, to September, 1884, as Instrumentman for the Juragua Iron Company, on the construction of railroads to mining property. From April, 1885, to November, 1886, he was employed as Instrumentman and Inspector on United States River and Harbor Improvements on Long Island Sound and Gedney's Channel.

---

[1] Memoir prepared by W. R. Devaul, Assoc. M. Am. Soc. C. E.

In February, 1887, Mr. Minor entered into private civil engineering practice in Greenwich, Conn., thereby realizing the ambition of every young engineer who has been working in various branches of the profession in order to gain the experience necessary to achieve his particular niche in the scheme of life. Starting his business career on a small scale in Greenwich, he advanced rapidly, due to his efficiency and close attention to detail, as well as to his honest business dealings with his fellow men. He became the Engineer for both the Town and Borough of Greenwich, having charge of the design of all highways, bridges, and sewers, in addition to private subdivision work and the laying out of large estates. From 1921 until Mr. Minor's death, the business was continued under the name of S. E. Minor and Company, Incorporated, with Mr. Joseph W. Cone as a partner.

Mr. Minor served for many years as Clerk and Treasurer of the Borough of Greenwich, was a member of the Board of Appeals of the Zoning Commission, and, for thirty-five years, acted as Engineer for the Greenwich Water Company and Port Chester Water-Works.

The passing of Mr. Minor is an occasion for deep regret on the part of a host of associates and friends throughout the State of Connecticut. All that is best in a sturdy New England heritage was typified in him; his integrity was unquestioned; his dealings with all were patient, fair, and sympathetic. As an Engineer, he ranked high in the profession, and strengthened his scientific training with practical judgment. He was always cheerful and even tempered; irritability and needless worry were foreign to his nature. Few men have had the unqualified esteem and respect of their friends that Sheldon E. Minor enjoyed.

In May, 1930, while still actively engaged in his work, a slight heart attack caused him to enter a private hospital in the Hotel Pennsylvania, New York City. He left the hospital after two weeks' treatment, in good spirits, and continued to devote himself to his work. Another attack caused him to return to the hospital on February 12, 1931, and, after seeming to improve, he suddenly grew worse and passed away on February 19, 1931.

Mr. Minor had served as a member of the Connecticut State Board of Civil Engineers as a Consultant on Dams since 1917; he had also been a member of the Connecticut Society of Civil Engineers since 1886 (President in 1913), the Yale Engineering Society, Chi Phi Fraternity, and of the Masonic Order.

He was married on December 6, 1887, to Mary E. Pierpont, of New Haven, who, with his son, Pierpont L. Minor, survives him.

Mr. Minor was elected a Member of the American Society of Civil Engineers on June 3, 1915.

HENRY MITCHELL, M. Am. Soc. C. E.[1]

Died December 1, 1902

Henry Mitchell, the son of William and Lydia (Coleman) Mitchell, was born at Nantucket, Mass., on September 16, 1830. He received his education in private schools and at home, the atmosphere of the latter being especially conducive to study. His mother had been a school teacher prior to her marriage; his father, a teacher and later a bank official and one of the Overseers of Harvard College, was an able astronomer who enjoyed a wide acquaintance among American men of science; a sister, twelve years his senior, was the well-known astronomer, Maria Mitchell, who during her brother's youth was busily engaged on astronomical work at home.

In 1849, Henry Mitchell entered the United States Coast Survey, his first assignment being in connection with triangulation. Later, he was transferred to the Hydrographic Branch, in which field his abilities soon became manifest. In 1854, entrusted with carrying out a tidal survey of Nantucket and Vineyard Sounds, he devised a tide-gauge for use on the open coast. During the next thirty-four years he was engaged in harbor surveys and in investigations in physical hydrography, the results of his investigations appearing in numerous papers, published principally in the Annual Reports of the Coast Survey between the years 1854 and 1888. He also served as Chief of the Department of Physical Hydrography.

In his harbor investigations Mr. Mitchell was concerned principally with studying the tide and current régime as related to hydrographic features. In these studies he was successful in elucidating the complex forces at work in maintaining or changing the channels or shore lines of the harbors in question. His principal publications are: "Tides and Tidal Pheuomena" (1868); "On the Reclamation of Tide Lands and Its Relation to Navigation" (1869); "Hints and Suggestions upon the Location of Harbor Lines" (1871); "On a Physical Survey of the Delaware River in Front of Philadelphia" (1878); "Physical Hydrography of the Coast of Maine" (1879); "Monomoy and Its Shoals" (1886); "The Circulation of the Sea Through New York Harbor" (1886); and "The Under-Run of the Hudson" (1888).

While still in his thirties, Mr. Mitchell became recognized as a leading Hydrographic Engineer and his services were utilized by various Government commissions. In 1866, he was directed by the Secretary of the Treasury at the nomination of the late Julius Erasmus Hilgard, M. Am. Soc. C. E., then in charge of the Coast Survey Office, to take the place of the late Alexander Dallas Bache, Hon. M. Am. Soc. C. E., on the United States Commission on Boston Harbor; and, subsequently, on application of the Massachusetts Board of Harbor Commissioners Mr. Mitchell became a member of its Advisory Council. In 1868, he was sent to Europe to investigate hydrographic and engineering progress, and while there he made a detailed inspection of the Suez Canal, upon which he prepared a report.[2]

---

[1] Memoir prepared by H. A. Marmer, M. Am. Soc. C. E.

[2] *North American Review*, October, 1869.

As Consulting Engineer, he served on various harbor boards in the United States. In 1874, he was appointed a member of the Commission on the Construction of an Oceanic Ship Canal, making a report of his inspection and investigation of the proposed routes through Nicaragua and the Isthmus of Darien.[3] That same year, President Grant appointed him a member of the Board of Engineers to survey the mouth of the Mississippi River, and in the following year, at the request of the late James B. Eads, M. Am. Soc. C. E., he served as a member of an Advisory Board in connection with the construction of the Mississippi Jetties. From 1875 to 1877 he served on the Advisory Board to the Harbor Commissions of Virginia and Rhode Island, and in 1879 he was appointed a member of the Mississippi River Commission by President Hayes.

In the world outside his chosen profession Mr. Mitchell's attainments likewise received wide recognition. Various scientific societies honored him by election to membership; in 1867, Harvard College conferred upon him the degree of Master of Arts; in 1869, he was offered the Professorship of Physical Hydrography at the Massachusetts Institute of Technology; and, in 1873, he was obliged on account of official duties to decline the same Chair in the Agassiz School of Science. In 1875, he was elected to the National Academy of Sciences.

In 1888, after thirty-nine years of service, Mr. Mitchell resigned from the Coast Survey. The following year, President Harrison offered him the Superintendency of the Survey, but his health did not permit him to assume the burdens of the office, and he declined the offer. Leading a retired studious life, he spent his summers in Nantucket and his winters near Boston, Mass. In 1890, he served as a member of the Commission of the Annual Assay of the Mint; in 1893, he was appointed a member of the Advisory Council of the World's Columbian Water Commerce Congress; and, in 1896, he published in the *Proceedings* of the American Academy of Arts and Sciences a biographical sketch of Ferdinand de Lesseps.

Mr. Mitchell was married three times. His first wife was Mary Dawes, of Boston, to whom he was married in his early twenties and who left him a widower after twelve years. In 1873, he was married to Margaret Hayward who died in 1875, about five months after the birth of a daughter, his only child. Two years later, he married his deceased wife's elder sister, Mary Hayward, who died in March, 1902. Mr. Mitchell died about nine months later at the home of his daughter in New York City.

Mr. Mitchell was elected a Member of the American Society of Civil Engineers on January 7, 1880.

JOHN MITCHELL MONCRIEFF, M. Am. Soc. C. E.[1]

DIED JANUARY 10, 1931

John Mitchell Moncrieff was born at South Shields, Durham, England, on March 8, 1865, the son of the late Mitchell Moncrieff. He was educated privately and served his apprenticeship at Messrs. Rennoldson's Engineering Works in South Shields.

---

[3] Coast Survey Report, 1874.

[1] Memoir prepared from information supplied by Thomas C. Finlay, Esq., London, England, and on file at the Headquarters of the Society.

After his apprenticeship Mr. Moncrieff entered the office of the late Mr. J. Watt Sandeman, of Newcastle-on-Tyne, England, as a pupil. Later, he became an Assistant in the same office, and, subsequently, joined Mr. Sandeman in partnership.

In 1906, this partnership was dissolved and, for many years, Mr. Moncrieff carried on an extensive practice by himself, at Newcastle-on-Tyne, specializing in bridges, docks, harbors, shipping facilities, and river-side improvements, generally.

One of his earlier works was the important undertaking of the new Redheugh Bridge across the River Tyne, at Newcastle, which was constructed according to his designs and under his supervision from 1899 to 1901, and which is considered to be one of the most difficult feats of bridge engineering ever achieved in England.

In 1907, when the Blackfriars Bridge disaster occurred Mr. Moncrieff was called in by Messrs. Sir William Arrol and Company to investigate and advise the Company concerning the causes of the accident. He frequently figured in other important cases of litigation and arbitration when he was called in to give expert engineering opinion, and also to advise in support of, or in opposition to, private bills before Parliamentary Committees of the House of Lords and the House of Commons.

In 1925, Colonel Moncrieff was invited by the Society for the Protection of Ancient Buildings to express his opinion for submission to the proper authority, on the question of the practicability of saving the structure of Waterloo Bridge over the Thames River. At this time he definitely expressed the opinion that the complete reconstruction of the damaged bridge piers was an absolute necessity and that this could be done only by taking down the bridge as a whole and likewise reconstructing it as a whole.

He was also associated with many other bridge undertakings during his career and was engaged on the design and construction or improvement of many large shipbuilding yards and graving docks in various parts of England. He was interested in railways, harbor developments, steel, timber, and reinforced concrete coaling staiths and their equipment, land development projects for industrial purposes, and the erection of large factories in steel and reinforced concrete, etc.

In June, 1917, he was appointed Director of Engineering Work in the Department of the Controller-General of Merchant Shipping at the Admiralty with the rank of Lieutenant-Colonel, Royal Engineers, and, in 1918, he had the honor of having the order of Commander of the British Empire conferred on him for his services.

In 1918, by permission of the Admiralty, Colonel Moncrieff resumed his practice for the purpose of designing important dock and harbor improvements at Falmouth, England, after which date he made London his headquarters.

In 1919 he took into partnership Mr. John W. Goldson, and his own son, Mr. Alan Moncrieff, with offices at Newcastle-on-Tyne and in London, and although this partnership was later dissolved, he continued to practice in London to the time of his death.

Colonel Moncrieff was a well-known member of many engineering societies in England and in other countries, among which are included the Institution of Civil Engineers, by which he was awarded the Telford Premium, 1893-94, and for which he was sometime Examiner in Theory of Construction; a sometime member of Council of the Northeast Coast Institution of Engineers and Shipbuilders, by which he was awarded the Engineering Gold Medal, 1905-06; a member of the Association of Consulting Engineers, serving as Chairman in 1924; and Immediate Past-President of the Institution of Structural Engineers, having held office in 1928-29 and 1929-30. At the time of his death, he was President of the British Section of the French Society of Civil Engineers, and a Vice-President of the newly formed International Association for Bridges and Structural Engineering, the administrative offices of which are at Zurich, Switzerland.

Colonel Moncrieff served on the Steel Research Committee formed by the Department of Scientific and Industrial Research, as well as many other committees and panels organized for research work. He wrote numerous engineering papers and contributed to the discussions on many others, all of which have appeared in the publications of the various engineering societies and in the engineering press. He was the author of "The Practical Column under Central or Eccentric Loads", which was published by the Society.[2]

At one time he was External Examiner in Civil Engineering to Sheffield University. As President of the Institution of Structural Engineers, Colonel Moncrieff was invited by the Ministry of Transport to organize the British Section of the International Congress for Reinforced Concrete which was held at Liege, Belgium, in September, 1930.

On June 4, 1890, he was married to Elizabeth, daughter of the late Mr. P. Allan, of South Shields, and is survived by his widow, two sons, and two daughters.

Colonel Moncrieff was elected a Member of the American Society of Civil Engineers on November 3, 1897.

GUY MOULTON, M. Am. Soc. C. E.[1]

Died December 3, 1929

Guy Moulton, the son of Emery and Mary J. (Churchill) Moulton, was born on February 25, 1861, in Cicero, N. Y.

He received his early education in Cicero, later enrolling at Cornell University, in Ithaca, N. Y., from which institution he received the degree of Bachelor of Science in 1881. Following his graduation, Mr. Moulton was employed by the Buffalo, Rochester, and Pittsburgh Railroad Company, serving as Stakeman, Draftsman, Transitman, and Leveler until the autumn of 1883.

---

[2] *Transactions*, Am. Soc. C. E., Vol. XLV (June, 1901), p. 334.

[1] Memoir prepared by Glenn D. Holmes, M. Am. Soc. C. E.

In the early summer of that year, he held the position of Assistant Engineer on location for the Pittsburgh and Allegheny Railroad.

From the autumn of 1883 to the spring of 1889, Mr. Moulton was engaged in other business, but returned to the Engineering Profession when he accepted the position of Transitman for the Lizard Creek Branch of the Lehigh Valley Railroad. In July, 1889, he was promoted to be Assistant Engineer in charge of construction on the Buffalo Extension under the late Paul S. King, M. Am. Soc. C. E., Chief Engineer.

Mr. Moulton left the service of the Lehigh Valley Railroad Company in December, 1892, to design and supervise the construction of an addition to the sewerage system at Watkins, N. Y., which work was successfully completed in the spring of 1893. Following a brief engagement with the New York and New England Railroad Company as Assistant Engineer, a similar position was accepted with a survey party for a private railroad in Tennessee during the summer of 1894. During the succeeding year he acted as Superintendent and General Manager for Charles McFadden in mining bituminous coal in Pennsylvania.

During the winter and spring of 1895-96 Mr. Moulton served as Assistant Engineer in charge of construction of the Jackson and Saginaw Railroad, in Michigan. In September, 1896, he assumed charge as Engineer and General Manager for McDonald and Sayre, General Contractors, on construction work of the Erie Canal Improvement. In November of the following year, he resigned to accept an appointment as First Assistant Engineer for the State of New York. He was in charge of the work from Syracuse, N. Y., to the Oneida County line during the "$9 000 000 Improvement", and after its completion, he was given charge of the entire Middle Division.

In 1904, as Resident Engineer, Mr. Moulton had charge of Residences 6 and 7, and a part of Residence 5, of the Barge Canal System. In 1909, he was appointed Division Engineer and, in 1911, First Resident Engineer of the System. In 1915 he was again made Division Engineer of the Barge Canal which position he held until his retirement in 1922.

From 1923 until his death he was engaged in consulting practice, maintaining an office in Syracuse. Mr. Moulton had been in failing health following a major operation in the summer of 1929, but continued his usual activity to the end. Death came to him as he was leaving his son's automobile on the way to his office, the result of a heart attack.

Mr. Moulton was known among the members of the Engineering Profession as a capable engineer of scrupulous integrity; to the public, as a trustworthy servant; among his associates, as a constant and unchanging friend; and to his family, as a devoted and generous husband and father.

He was a member of the First Universalist Church, which he served in many ways. He was affiliated with the Masonic bodies and was a member of the Citizens' Club of Syracuse. As an active member of the Syracuse Technology Club, he served as Director, Vice-President, and President, and on many of its committees. Mr. Moulton ably served his city upon two Flood Prevention Committees when appointed in 1915 and 1925 to study flood hazards and remedies in Syracuse.

He was a great lover of Nature. During the early morning and evening hours he took especial delight in his garden and the growing of roses, dahlias, gladioli, and other flowers.

He was married on March 16, 1887, to Sarah Adeline Wright, of Clay, N. Y., who, with three sons, Webster C., Lloyd W., and Guy W. Moulton, survives him.

Mr. Moulton was elected a Member of the American Society of Civil Engineers on March 1, 1905.

ANDREAS WENDELBO MUNSTER, M. Am. Soc. C. E.[1]

Died May 13, 1929

Andreas Wendelbo Münster was born in Bergen, Norway, on December 26, 1853, the son of Lt.-Col. Frederik Julius Münster, of the Norwegian Army, and Grete Bull Münster, a sister of the famous violinist, Ole Bull. Mr. Münster received his technical education at the Chalmarske Institute at Gothenborg, Sweden, from which he was graduated in 1873.

In August following his graduation, Mr. Münster entered the service of the Norwegian Government Railroads, where he remained for eight years, holding, successively, the positions of Assistant Engineer on location until the fall of 1876; Assistant Engineer on construction until 1880; and Resident Engineer on construction until his resignation in May, 1882.

On his arrival in the United States in 1882, Mr. Münster went West and in July accepted the position of Assistant Engineer on the Rocky Mountain Division of the Northern Pacific Railroad Company, with headquarters in Helena, Mont. After the completion of the road in 1883, he spent the following winter and spring as Draftsman in the Office of the Surveyor-General of Montana and, in August, 1884, was appointed Bridge Engineer in the City Engineer's Department at St. Paul, Minn. Among the more important structures which were designed by him and erected under his supervision were several bridges across the Mississippi River in St. Paul, Minn., including the Robert Street Bridge, an iron bridge, 1 540 ft. long, which was completed in 1886; High Bridge, an iron structure, 2 770 ft. long; and the Wabasha Street Bridge, also an iron structure; also, the Third Street Viaduct, an iron and masonry structure, 1 900 ft. long, completed in 1887; and the Colorado Street Stone Arch Bridge, a skew arch of 70 ft. span.

Mr. Münster had charge of the many artesian wells owned by the City of St. Paul as a part of its water supply and he also built the Aberdeen Hotel, on the corner of Dayton and Western Avenues, which, at one time, was considered one of the finest buildings in that city.

In April, 1904, he was appointed Chief Engineer of the Great Western Railway Company. He continued in that capacity until his resignation in July, 1906. During his connection with this Company, Mr. Münster designed and supervised the construction of the following: Grain Terminals and Independent Elevator at Omaha, Nebr.; the Red Wing, Minn., Depot; the Great

---

[1] Memoir compiled from information supplied by Charles F. Loweth, Past-President, Am. Soc. C. E., B. Thomas, Civ. Engr., Seattle, Wash., and on file at the Headquarters of the Society.

Western Elevator at Kansas City, Mo.; the Fort Dodge, Iowa, Depot; shops, store rooms, and a forty-stall round-house, at Oelwein, Iowa, etc. He also was in charge of the double-tracking of the Chicago Great Western Railroad from Stockton to Galena Junction, Ill.

In 1906, Mr. Münster engaged in private practice as a Consulting Engineer with offices in Seattle, Wash., which he continued until his death. Because of his enviable record in St. Paul, his services were soon sought throughout the West. He was Consulting Engineer for the Chicago, Milwaukee, and St. Paul Railway Company during the construction period of the Western Extension to Seattle (1908 to 1911), and designed several of the railway terminals and division structures in Washington and Idaho, which included docks, wharves, and freight houses in Seattle and Tacoma. He was also Consulting Engineer for the Great Northern Railway Company, and among the many structures which he designed for this road was the reinforced concrete dock in Vancouver, B. C., Canada.

From 1914 to 1923 Mr. Münster served as Consulting Engineer for the City of Seattle, on the design and construction of several of its bridges, notably those at 15th Avenue West, Fremont Avenue, Eastlake Avenue, and West Spokane Avenue. From 1923 until his death, he also acted as Chief Engineer of the Bridge Department for that city. The Montlake Bridge, over the Lake Washington Ship Canal, is an outstanding structure which was designed and constructed under Mr. Münster's supervision. On its completion his friends in Seattle arranged a celebration in his honor.

A nobleman by birth, Mr. Münster was a quiet and dignified gentleman, who commanded the admiration of all who knew him. Through him as a man of eminence in his profession is added greater renown to those engineers of Norwegian birth who have practiced their profession in the United States. His work which was of high quality prompted by a noteworthy pride in professional achievements, was ever conscientiously performed. Many of the bridges that he designed forty years or more ago are still giving satisfactory service. They are models of fine design which will stand as monuments to his unusual skill and ability as an engineer. The character of his work is a tribute to him and accounts for the esteem in which he was held by the City of Seattle.

On September 17, 1926, Mr. Münster was married to Clara von Tangen Sullivan who survives him, together with a sister, Mrs. Storm Bull, of Madison, Wis.

Mr. Münster was elected a Member of the American Society of Civil Engineers on May 1, 1889.

RALPH WALDO NICKERSON, M. Am. Soc. C. E.[1]

DIED MAY 13, 1929

Ralph Waldo Nickerson, of an old New England family, was born on November 26, 1881, at Tiverton, R. I. His father was Capt. George Ferdinand

---

[1] Memoir prepared by J. E. Wadsworth, J. B. French and H. G. Balcom, Members, Am. Soc. C. E.

Nickerson, and his mother, Marie (Springer) Nickerson, was a lineal descendant of Knight Springer of Revolutionary Service.

Mr. Nickerson prepared for college in the public schools of Tiverton. He entered Worcester Polytechnic Institute, Worcester, Mass., in the fall of 1899 and was graduated in 1903 with the degree of Bachelor of Science. Immediately after his graduation he entered the employ of the American Bridge Company as a Structural Draftsman in the East Berlin, Conn., Plant. He was promoted to the position of Squad Leader in 1906, and had supervision of a squad of draftsmen preparing detail drawings of bridges and buildings. In 1911 he was again promoted, this time to the position of Designing Engineer in the New York Office of the Company.

As a Detail Draftsman he had charge of the detailing of some of the most important structures fabricated by the Company, and as a Designer he designed the steel framework for many large manufacturing plants, among which were the new plants of the American Brass Company at Waterbury, Ansonia, and Torrington, Conn., and Kenosha, Wis. He also had general supervision of the detailing of his own designs.

During his connection with the American Bridge Company Mr. Nickerson was released to do special work for H. G. Balcom, M. Am. Soc. C. E., Consulting Engineer of New York City, under whose direction he prepared designs for several large office buildings there.

In December, 1924, he resigned his position with the American Bridge Company and opened an office in New York City, as Consulting Engineer, making a specialty of structural steel, reinforced concrete, and foundations. While in private practice Mr. Nickerson designed the structural steelwork and foundations for a large number of theatres, office buildings, and miscellaneous structures, in various parts of the country, and established a reputation for conscientiously thorough and high-class work.

In 1928 Mr. Nickerson entered the Engineering Department of the George A. Fuller Company and remained with that Company until his death on May 13, 1929. During this period he had direct supervision of the work of the Company on the No. 400 Madison Avenue Building, the Beaux Arts Apartments (312 and 313 East 44th Street), and the Astor Apartment (520 East 86th Street), all in New York City.

Mr. Nickerson was a man of sterling character and of indisputable integrity. He was an indefatigable worker and a man of unquestioned reliability. He had the confidence of the companies by which he was employed and of his clients for whom he acted as a Consultant.

He was a member of the Union County (New Jersey) Chapter of the Society of Professional Engineers and Land Surveyors, the Sons of the American Revolution, Elizabethtown, Pa., Chapter; and the Architects and Engineers Square Club. He was also a member of Hillside (N. J.) Lodge No. 241, F. and A. M., and an Elder in the Hillside Presbyterian Church.

He is survived by his widow, Edith North Nickerson, and two sons, Ralph Waldo and Stanley North Nickerson.

Mr. Nickerson was elected a Member of the American Society of Civil Engineers on April 20, 1925.

GEORGE H. NORMAN, M. Am. Soc. C. E.[1]

Died February 4, 1900

George H. Norman was born in Newport, R. I., on January 1, 1827, the son of George W. and Hannah Norman, and maintained his home there throughout his life.

His education was limited to that provided by the common schools of Newport. He left school at an early age and began work in a shoe store. Before he was twenty years old, he had acquired the business, on the death of his employer, and continued it successfully for some time.

In 1849, Mr. Norman, in partnership with Mr. William H. Cranston, purchased the *Newport Daily News,* which they continued to publish until 1856. A gas company was started in Newport early in the Fifties and the promoter, desiring to secure the aid of the newspaper, interested Mr. Norman in the matter to such an extent that he took some stock in the Company, became its Secretary, and acted for some time as its Bookkeeper and Cashier. This was his introduction to public utilities.

Mr. Norman's first venture in the construction field, in which he afterward became so successful, was the installation of gas-works at Waterbury, Conn. With the experience gained on this work, and in company with a fellow townsman, Mr. William Fludder, Mr. Norman in 1857 undertook the construction of a gas plant in Cuba—the St. Fago de Cuba Gas-Works. He went to England to purchase pipe and other material, and thence to Cuba to construct the works. Later, Mr. Norman entered the field of general engineering contracting. The construction of sewers in Manchester and Nashua, N. H., was among his earlier works.

Subsequently, he turned his attention to the construction and operation of water-works. About 1876, he began the Newport Water-Works, organizing a corporation to build and operate the plant. He continued in control of this Company until his death, and the ownership remained in his family until about 1929.

After building the water-works at Newport, Mr. Norman organized water companies and built works in many other places, and, also, as contractor, constructed extensions to a number of existing municipal water-works. Most of this work was done between 1880 and 1890, although some of it was continued as late as 1898. Among the places for which works were built or extended, may be mentioned the following: Manchester, N. H.; Newburyport, Lowell, Beverly, Gloucester, Lynn, and Franklin, Mass.; Bristol, Warren, and Jamestown, R. I.; Middletown, Conn.; Seneca Falls and Hoosick Falls, N. Y.; Frankfort, Ind.; and Green Bay and Fort Howard, Wis. During this period he was also engaged in railroad construction, both in New England and in Texas.

A large proportion of the distribution pipe in these water-works systems was of sheet iron, lined and surrounded with cement mortar, a type of construction which was then at the height of its popularity. Mr. Norman's skill in construction and laying pipe of this type at comparatively low cost, and his business acumen in establishing water companies in growing towns and retaining the control of them until they could be sold at a good profit, were probably responsible in large measure for the financial success which he achieved.

---

[1] Memoir prepared by Charles W. Sherman, M. Am. Soc. C. E., chiefly from information furnished by Bradford Norman, Esq., Newport, R. I.

Mr. Norman was married on May 23, 1859, to Abby Durfee Kingsley, daughter of Rufus B. Kingsley, of Newport. He had six sons and three daughters.

Shortly after the re-organization of the American Society of Civil Engineers, Mr. Norman was elected a Member on February 17, 1869; and on March 23, 1870, he became a Fellow, by contributing a substantial sum to the permanent funds of the Society. In 1872, he endowed the first prize established by the Society, the Norman Medal, which is awarded annually for a paper by a member of the Society, "which shall be judged worthy of special commendation for its merit as a contribution to engineering science."

GEORGE HARVEY NORTON, M. Am. Soc. C. E.[1]

DIED MARCH 4, 1930

George Harvey Norton, the son of George Harvey Norton, M. D., and Martha Graves Norton, was born in East Pembroke, N. Y., on October 24, 1863. In 1887, he was graduated from Cornell University, at Ithaca, N. Y., with the degree of Civil Engineer.

In his early life, during 1887 and 1888, Mr. Norton was engaged in survey work on the Duluth, South Shore, and Atlantic Railroad and the Chicago and West Michigan Railroad. While thus engaged, he went to Buffalo, N. Y., on a vacation and obtained a position as Substitute Draftsman with the Grade Crossing Commission. Within six months, he was appointed Assistant Engineer, under the late George E. Mann, M. Am. Soc. C. E., who for years served both as Engineer for the City of Buffalo and the Grade Crossing Commission. Mr. Norton entered the service of the City of Buffalo on November 1, 1888, and was closely associated with all its more important engineering projects from that time until his death.

Among his notable accomplishments, were the improvement of Scajaquada Creek and, coincident with this, the improvement of the Buffalo River and Cazenovia Creek, for relief from floods. From 1889 to 1895, as Assistant Engineer, he also constructed and operated a temporary sewage pumping plant, made paving plans, and had charge of the underground construction preceding paving, on about 150 miles of streets. During 1896 and 1897, he was engaged in paving construction.

From 1898 to 1908, Mr. Norton, as Assistant City Engineer, was in immediate charge of all bridge, harbor, and hydraulic work of the city (except grade-crossing elimination), covering both construction and maintenance. including the maintenance of many grade-crossing structures after completion. He was in charge of the design and construction of several fixed bridges, having spans of from 40 to 150 ft., and the general location, design, and construction of temporary structures and foundations for two bascule bridges, one of 110-ft. span and the other of 166-ft. span, together with the supervision of the construction and operation of these bridges.

---

[1] Memoir prepared by Edward P. Lupfer, M. Am. Soc. C. E.

His name is likewise outstanding in the development of Buffalo's waterfront. He had immediate supervision of the plans and the dredging of Buffalo's Inner Harbor, this project costing approximately $900 000, and, during 1908, he was engaged in survey and engineering work in negotiations for the improvement of a large part of Buffalo's Outer Harbor, as well as in making hydraulic studies and plans for flood abatement and extending the navigable channel in Buffalo River, at a total cost of about $1 000 000.

From 1909 to 1923, as City Engineer, Mr. Norton was in charge of all street, sewer, sidewalk, and bridge construction and maintenance, all city harbor deepening and extension, land surveys, the supervision of franchise, occupation of streets, and valuation of physical properties therein, as well as a great variety of other engineering work arising in a large city. Some of the larger projects included the improvement and extension of navigation; the beginning of the Scajaquada Creek drain, one of the largest of such projects for placing streams below ground; water-front improvements; studies of sewage treatment; and the settlement of long disputed water-front rights and titles. Expenditures in this work, under the City Engineer, averaged about $3 000 000 per year.

As City Engineer, Mr. Norton also was made Chairman of the City Planning Committee, and served as such until the spring of 1926. During this time there was developed a general City Plan, including the location of public buildings and the adoption of a zoning ordinance. He was also a Director of the Buffalo City Planning Association and Niagara Frontier Association, and was deeply interested in general city planning.

In 1923, Mr. Norton became Chief Engineer of the Grade Crossing and Terminal Station Commission, and served in this capacity until his death. His career in this office was a notable one, and as Chief Engineer of the Grade Crossing Commission, his services in connection with the location and design of the New York Central Terminal at Buffalo were, perhaps, the crowning achievement of a distinguished career.

His interests and activities were many and varied, and in addition to his civil life, he had an honorable military record. He enlisted in the Sixty-fifth Infantry, New York National Guard, in 1889, advancing to Corporal, Sergeant, Second and First Lieutenant, and, in May, 1898, he received his Commission as Captain, in which rank he served throughout the Spanish-American War. He was placed on the Reserve List in 1918, re-assigned to the Sixty-fifth Field Artillery in 1919, and appointed Major of Field Artillery and placed on the Retired List in 1920.

Major Norton's services to the Buffalo Chamber of Commerce as Vice-President and as member of the Board of Directors for two consecutive terms, were long and useful. He was particularly active on the City Planning and

Marine Airport Committees, and developed the first plans for the landing area at the foot of Georgia Street, Buffalo.

He was the author of various articles for technical papers and social publications. He was a Past-President of the Buffalo Section of the American Society of Civil Engineers, the Engineering Society of Buffalo, and the American Society for Municipal Improvements. He was a member of the City Planning Institute and the Cornell Society of Engineers.

In civil and social organizations of the city Major Norton also was active. He was a member of many prominent groups, some of which are as follows: The Automobile Club of Buffalo, the Scottish Rite Masons, the Shrine, the Rotary Club, the Torch Club, the University Club, the Buffalo Athletic Club, and the Hook and Axe Club. He was a member of the Central Presbyterian Church.

Major Norton's life was cut short at the crest of his active career, when he passed away on March 4, 1930. He was one of the finest of men, a worker who loved his work and who sought no ostentatious praise. He was filled with the spirit of service to the community, and possessed a kindly, gentle character that endeared him to all who knew him. He was a highly esteemed gentleman, and, in his death, Buffalo has lost one of its great builders. He was deeply beloved by all and his passing leaves a void that can not easily be filled.

At the time of his death, fitting testimonials to Major Norton's sterling qualities appeared in many publications in his own section of New York State, typical of which is the following from the *Buffalo Evening News*:

"The death of Major George H. Norton removes a man who has had a part in nearly every big public improvement that has been carried out in Buffalo during the last forty years. In many of them he was the directing mind. As City Engineer and, later, as Engineer for the Grade Crossing Commission, he had responsibilities far beyond any credit which his modest mind ever thought to claim.

"Moreover, it may be said for him that in a long lifetime of public employment, he retained the confidence and the friendship of those with whom he was associated to such a degree that even dissent from his plans never meant personal criticism. Closely associated as he was with government, he was regarded by everybody as distinctly an engineer, not a politician."

He is survived by his widow, Mrs. Elvina Margaret Schiferle Norton, and a son, George H. Norton, Jr.

Major Norton was elected a Member of the American Society of Civil Engineers on April 7, 1915, and at the time of his death was serving on the Board of Direction.

CHARLES WELLMAN PARKS, M. Am. Soc. C. E.[1]

Died June 25, 1930

Charles Wellman Parks was born at Woburn, Mass., on March 22, 1863, the son of Granville and Elizabeth Augusta (Carter) Parks. His ancestors were of early Colonial New England stock. He was graduated from the Woburn High School in 1879, and, in 1929, he attended the Fiftieth Anniversary re-union of his class. Entering Rensselaer Polytechnic Institute, at Troy, N. Y., in 1879, he was graduated in 1884 with the degree of Civil Engineer. In 1899, he was graduated from the Columbian University, (now George Washington University) with the degree of Bachelor of Laws.

---

[1] Memoir prepared by Glenn S. Burrell, Capt. (C. E. C.), U. S. N., M. Am. Soc. C. E.

Mr. Parks' first practical engagement was in the fall of 1881 as Instrumentman on the Missouri River Survey with headquarters at Council Bluffs, Iowa. From December, 1881, to April, 1882, he was Leveler on the Arkansas Northern Railroad, and from April to August, 1884, he served as Engineer with the Denver, Memphis, and Atlantic Railway Company in Kansas. For the next four months he was employed as Engineer on railroad work in Colorado. In 1885 he was engaged in designing street railway equipment in Boston, Mass.

In September, 1885, Mr. Parks became Assistant to the Director of the Rensselaer Polytechnic Institute, and Engineer with the Electric Manufacturing Company of Troy, the first company in the United States to manufacture electrical testing instruments. From September, 1886, to September, 1894, he was the head of the Department of Physics at Rensselaer Polytechnic Institute.

From November, 1888, to March, 1890, Mr. Parks served under United States Government appointment as Superintendent of Liberal Arts at the Paris Exposition of 1889. The exhibit of the Rensselaer Polytechnic Institute under his charge was awarded the Grand Prix. He was made Officier de l'Instruction Publique of France, and awarded the decoration with golden palms. He also served under Federal appointment as Special Agent to the World's Columbian Exposition in Chicago, Ill., in 1893. As Special Agent of the United States Bureau of Education, he visited and reported on the educational exhibits at the expositions at Antwerp, Belgium, Lyons, France, Genoa, Italy, and Zurich, Switzerland, and investigated and reported on forestry schools and systems in various European countries.

In 1895 and 1896 he was engaged on the New York State Land Survey. In July, 1896, Mr. Parks became an Examiner in the United States Patent Office and, at the same time, pursued the course in law at Columbian University. In the summer of 1897, yielding to the persuasion of the late Rear-Admiral Mordecai T. Endicott, C. E. C., U. S. N., Past-President, Am. Soc. C. E., then Chief of the Bureau of Yards and Docks, United States Navy Department, Mr. Parks took the competitive examination for entrance into the Corps of Civil Engineers, United States Navy. Commissioned a Civil Engineer with the rank of Lieutenant (Junior Grade) in July, 1897, he was promoted to the rank of Lieutenant in February, 1901, Lieutenant Commander in October, 1909, Commander in August, 1913, Captain in July, 1917, and Rear-Admiral in January, 1918.

From July, 1897, to February, 1898, Lieutenant Parks was on duty at the Navy Yard, Norfolk, Va. From that time until the spring of 1899 he was on duty in the Bureau of Yards and Docks, Washington, D. C. During the major part, if not all, of this period he was the only commissioned Assistant to the Chief of the Bureau. This covered the period of stress incident to the Spanish-American War. During this period he designed the naval dry docks for Portsmouth, N. H., and Boston, Mass., including the first electric pumping plants used for United States Naval dry docks.

Thereafter, he served as Officer-in-Charge of the design, construction, and maintenance of public works at various stations as follows: San Juan, Porto Rico, 1899-1902; the Navy Yard, Boston, 1902-1905; and the Navy Yard, Portsmouth, 1905-1908. At Portsmouth he was charged with making the material preparations for the Russo-Japanese Peace Treaty sessions, for which he earned high commendation.

In May, 1908, Lieutenant Parks was selected by the Secretary of the Navy to take charge of the development of Pearl Harbor, Hawaii, for Naval purposes. Surveys, triangulation, soundings, and borings were undertaken at once. At the time of his detachment in February, 1910, the dredging of the 4½-mile entrance channel had been under way for about a year and excavation for the dry dock was also well along. He was subsequently detailed, 1910 to 1912, as Inspector of material and apparatus for the Navy at the works of the General Electric Company, at Schenectady, N. Y. Following this, he was Public Works Officer at the Navy Yard, Philadelphia, Pa., from 1912 to 1914.

In December, 1914, Commander Parks was again selected by the Secretary of the Navy as Public Works Officer for the Naval Station, Pearl Harbor. A very important part of the assignment was the resumption of the construction of the 1 000-ft. dry dock by contract, with radical changes in the design and method of construction. Concurrently, there was active development of the remainder of the station, including shops, storehouses, roads, railroads, coaling plant, fuel oil plant, radio station, ammunition depot, water-front, hospital, etc. The dry dock was brought to practical completion by the time of his detachment in January, 1918.

In the meantime, he had been promoted to the rank of Captain. In January, 1918, in the great stress of the wartime building program, he was commissioned Chief of Bureau of Yards and Docks (and Chief of the Corps of Civil Engineers), with the rank of Rear-Admiral. Some conception of the magnitude of the work handled by the Bureau in 1918 may be had from the fact that the expenditures for construction work were about forty times as great as in peace time preceding the war.

There was a vast expansion of industrial and storage facilities, of dry docks and shipbuilding facilities, of training stations, air stations, submarine bases, hospitals, and ammunition depots. In the meantime the Bureau was carrying on a vast amount of construction work in Europe, including air stations in Great Britain and France, and the world's greatest radio station at Croix de Hins, France. In addition, the Bureau was assisting many commercial shipbuilding and repair plants in the expansion of their facilities to do Navy work.[2]

---

[2] For a full account of this work the interested reader is referred to the book, "Activities of the Bureau of Yards and Docks in the World War", of which Admiral Parks was Editor-in-Chief.

On the conclusion of the World War, Admiral Parks was confronted with another difficult task, that of slashing budgets, reducing construction programs, and decreasing forces at the Bureau and at various stations. It is one thing to go with the tide, but another to check it. This task he performed vigorously with a great saving of taxpayers' money.

During his career, Admiral Parks served on numerous boards of officers making investigations and reports on important Naval projects, including dry docks, coaling stations, the layout of the Pearl Harbor Station, power plant consolidations, and similar construction work. He was a member of various boards on the admission and promotion of officers in the Corps of Civil Engineers. In 1905, he was ordered to Europe to make an investigation and report on progress in the manufacture of direct-current turbo-generators.

In recognition of Admiral Parks' outstanding service as Chief of Bureau during the war he was awarded the Distinguished Service Medal (Navy) by the United States Government. The Republic of France made him a Commandeur Legion d'Honneur, this reward being conferred by General Nivelle. The degree of Doctor of Laws was conferred upon him by George Washington University, on February 22, 1921.

In December, 1921, Admiral Parks was found to be suffering from chronic heart disease, and he was thereupon retired for disability incident to the service, with the rank of Rear-Admiral. In 1922, he gave his services to the United States Department of Justice, assisting with the handling of cases growing out of war-time construction contracts. He continued in this work for nearly eight years until shortly before his death. His long and broad experience at the Bureau and various stations, coupled with his legal and engineering education, especially fitted Admiral Parks for this work. With his keen analytical mind he was of the greatest assistance to the Department of Justice in handling the various cases. He was offered active duty for this work, but preferred to carry on in his retired status with no increase in compensation.

Admiral Parks was a Fellow of the American Institute of Electrical Engineers, and a member of the following societies: the Rensselaer Society of Engineers; the American Society of Naval Engineers; the Society of American Military Engineers; the National Geographic Society; the American Geographical Society; the American Association for the Advancement of Science; and the New England Genealogical and Historical Society.

During the latter forty years of his life Admiral Parks was deeply interested in genealogy, and made exhaustive studies of the Parks family in America and in England. This may be said to have been his one hobby.

He was also a member of the following clubs: The Algonquin (Boston), the Cosmos and Army and Navy (Washington), and the Army and Navy of America (New York). He was a member of the Presbyterian Church for fifty years.

Admiral Parks was married in Troy, on April 14, 1887, to Martha Bessac Frear, of Coxsackie-on-the-Hudson and Troy, who survives him. He is also survived by three brothers, Granville H. and Karl E., of Washington, and Albert F. Parks, of South Norwalk, Conn.

On June 25, 1930, Admiral Parks died of pneumonia and heart trouble at the Naval Hospital in Washington, after an illness of several months. He was buried at the National Cemetery at Arlington, Va., with full Naval honors for his rank.

In the death of Admiral Parks the Society has lost an illustrious member. His patriotism, all-round professional ability, untiring zeal, and executive ability combined to make him a most valuable public servant in peace and in war. He always maintained the courage of his convictions and was indefatigable in protecting the Government's interest. He was ever ready with counsel for those who sought his advice.

Admiral Parks was elected a Member of the American Society of Civil Engineers on October 3, 1906.

JAMES SUYDAM POLHEMUS, M. Am. Soc. C. E.[1]

Died September 29, 1930

James Suydam Polhemus was born in Astoria, N. Y., on March 26, 1852. He was the fifth to bear the same name, born in direct succession in the United States, the family having originally come from Holland. His father was engaged in the shipping and commission business in New York, N. Y.

Mr. Polhemus' boyhood was spent very largely on the waters of the East River and Long Island Sound and his interests naturally followed along marine lines. After the usual preliminary school education, he attended Lehigh University, from which institution he was graduated in Civil Engineering in 1872. He was a classmate of Dr. Drinker, President Emeritus of the University.

As a young man, Mr Polhemus was somewhat delicate and at the end of his Junior year at the University, in June, 1870, his father sent him on a trip around the world. He sailed from New York in one of the old-style Pacific Mail steamers of that time, and proceeded by way of the Mediterranean and Suez Canal to China. His account of the voyage made in those early days was most interesting, as he was a close observer and possessed of a most retentive memory. On his arrival in China, after stopping in Japan, he was advised of his father's death, and he returned directly to New York by way of San Francisco.

Shortly after his arrival in New York, Mr. Polhemus was employed on the construction of the Albany Water-Works. He re-entered Lehigh University in the Class of 1872, and after graduating he became connected with river and harbor work under the Corps of Engineers, United States Army. His first duty was at Galveston, Tex., later at New Orleans, La., and his work consisted principally in surveying the many bays and bayous adjoining the Gulf of Mexico in Texas and Louisiana. The conditions under which he carried out his work were most strenuous, for everything in that section was very primitive in those days when one found himself away from the cities or regular settlements.

---

[1] Memoir prepared by Frederick C. Schubert, M. Am. Soc. C. E.

After three years in the Gulf region, Mr. Polhemus was transferred to the United States Lake Survey, which was then making the first hydrographic survey of the Great Lakes. He was more particularly connected with the Computing Section of the Astronomical Division, which was determining the precise latitude and longitude of the various stations. Leaving the Lake Survey in 1877, he was assigned to the Wheeler Survey, then doing exploration work in California. He returned to Galveston in 1878, where he remained until 1880, when he was transferred to Portland, Ore.

After his arrival in Oregon, Mr. Polhemus was intimately connected with many river and harbor improvement works, more especially at Yaquina and Coos Bay, where he initiated and carried on the work on the jetties at the entrances to these harbors. He was a pioneer in this class of work, as the jetties at Yaquina Bay were the first of the kind on the Pacific Coast.

Mr. Polhemus retired from active work on March 25, 1924, after a long and honorable record of fifty-two years of continuous Government service. His opinion and judgment were highly valued by all. No single individual possessed a greater knowledge or fund of information regarding the entire Western Coast. His information was not confined to engineering matters, and he could discuss almost any subject. He was a great reader and kept himself fully abreast of the world's progress along every line of effort.

Mr. Polhemus had suffered from heart trouble for several years, and a recurrence of this trouble caused his death after a week's severe illness. He died at his home in Portland on September 29, 1930.

Personally, he was a most lovable character, even tempered, and kindly. News of his passing was heard with the greatest regret up and down the entire Western Coast, for it is safe to say that no one possessed more friends in all walks of life.

Shortly after his arrival in Oregon, Mr. Polhemus was married to Mary Clark Daly, daughter of the late W. A. Daly. Mrs. Polhemus died in 1922. A son, James H. Polhemus, now Manager of The Port of Portland, and a daughter, Harriette, survive their father.

Mr. Polhemus was elected a Member of the American Society of Civil Engineers on October 3, 1894.

JAMES BOND POPE, M. Am. Soc. C. E.[1]

DIED JULY 13, 1928

James Bond Pope, the son of Algernon R. and Nannie L. (Walker) Pope, was born on January 29, 1866, in Macon, Mo.

Mr. Pope began his engineering career in the early Eighties on the construction of the St. Louis-San Francisco Railroad and, in 1887, was employed by the California Southern Railroad Company. In 1889 and 1890, he located and built roads and railroads for lumber companies and, in 1891 and 1892, was Locating Engineer for the Arrowhead Reservoir Company.

---

[1] Memoir prepared by George W. Rear, J. L. Campbell, and H. M. Mitchell, Members, Am. Soc. C. E.

In the fall of 1892 until the spring of 1902, Mr. Pope was engaged in private practice at San Bernardino, Calif. At this time he had charge of the location, design, and construction of several water supply systems for irrigation and domestic use. Among these were the extension of Bear Valley Irrigation System, the Highland Domestic Water Company, and the domestic water supply and irrigation system of Colton, Calif.

In 1902, Mr. Pope entered the service of the Southern Pacific Railroad Company and until June, 1907, he was engaged in reconnaissance and office work under the late William Hood, M. Am. Soc. C. E., Chief Engineer. In June, 1907, Mr. Pope took up the work of physical valuation of railroad properties, devoting two years to those of the Oregon Railroad and Navigation Company which at that time was affiliated with the Southern Pacific Company as part of the Harriman Lines. He became Valuation Engineer on the Southern Pacific Railroad Company's own properties in the fall of 1909, continuing as such until September, 1922, when he became Consulting Engineer of Valuation.

In June, 1926, Mr. Pope resigned his position with the Railroad Company and opened an office as a Consulting Valuation Engineer. He was one of the pioneers in railroad valuation and had a prominent and important part in the development of its theories and in the organization of valuation forces as required by a large railroad company. He was peculiarly fitted to help solve the many complicated and difficult problems which arose from day to day in this new phase of railroad engineering work, and his passing has been keenly felt by those associated with him for so many years.

He was a member of several engineering societies and clubs and an active member of the Masonic Fraternity.

He was married in 1890 to Annie Florence Bannon, a native of England, who survives him. He is also survived by his son, Fred B. Pope, and a grandson, James B. Pope, of Stockton, Calif.

Mr. Pope was elected a Member of the American Society of Civil Engineers on February 4, 1914.

GEORGE FREDERICK PORTER, M. Am. Soc. C. E.[1]

DIED SEPTEMBER 19, 1930

George Frederick Porter, the son of Frederick B. and Helen C. (Perry) Porter, was born at Detroit, Mich., on October 7, 1862.

His education was received in the public schools of Detroit; from training in general engineering work in the office and employ of Mason L. Brown, Civil Engineer and Surveyor, in Detroit, from 1885 to 1894; as Draftsman and, later, as Engineer in the employ of the Detroit Bridge and Iron Works, 1894 to 1896; with the Keystone Bridge Company of Pittsburgh, 1896 and 1897; with the Detroit Bridge and Iron Works again from 1897 to 1900; and from his own unwearying study of engineering matters in his chosen field.

---

[1] Memoir prepared by the following Committee of the Detroit Section: C. M. Goodrich, F. C. McMath, Willard Pope, and C. W. Hubbell, Members, Am. Soc. C. E.

In the main, self-educated, his outstanding ability was fittingly recognized in October, 1921, when McGill University conferred upon him the honorary degree of Doctor of Laws.

In 1900, The Canadian Bridge Company, Limited, of Walkerville, Ont., Canada, was formed, and Mr. Porter was one of the small group which launched this enterprise, himself taking charge of the planning and construction of the plant. His connection with this Company was unbroken until 1909 when, because of his outstanding ability in the detail planning of large structures, his services were demanded by the Board of Engineers appointed by the Government of the Dominion of Canada to design and supervise the construction of the Quebec Bridge, which now spans the St. Lawrence River near the City of Quebec, Que., Canada.

Shortly after the completion of the official design by the Dominion Board of Engineers in June, 1910, The Canadian Bridge Company, Limited, and the Dominion Bridge Company, Limited, of Montreal, Que., Canada, formed the St. Lawrence Bridge Company to tender bids upon the Quebec Bridge, and Mr. Porter was given charge of working out the detail plans for a new design for this structure. The contract was awarded upon the basis of this alternative design, and Mr. Porter was made Engineer of Construction, which position he held, supervising not only the work of detail design and fabrication, but that of erection as well, until the completion of the structure in 1918 and the dissolution in 1919 of the Company formed to undertake it. He is entitled to a large share of the credit for the success of this huge engineering feat.

Mr. Porter, G. H. Duggan, M. Am. Soc. C. E., and the late Phelps Johnson, M. Am. Soc. C. E., were joint authors of the paper entitled "The Design, Manufacture and Erection of the Superstructure of the Quebec Bridge", presented in 1919 to the Engineering Institute of Canada,[2] a paper the unusual merit of which was rewarded by the award of the Gzowski Medal.

In 1919, Mr. Porter returned to the Canadian Bridge Company, Limited, as Chief Engineer, subsequently becoming Consulting Engineer. During this period many interesting and important engineering problems came under his immediate supervision. Among these should be mentioned the fabrication and erection of six of the 200-ft. lift spans over the Welland Ship Canal, and the fabrication and launching of the nine steel tubes, each 31 ft. in diameter and 248 ft. long, for the Detroit and Windsor Vehicular Tunnel under the Detroit River. With the close of 1929, he retired from active service with this Company, retaining, however, the keen interest in the engineering world which he had always shown. Happily his later years were marked by sufficient leisure to enable him to tour extensively on the American Continent, to visit the Hawaiian Islands, and to visit and examine the works of the Panama Canal.

Mr. Porter's interests were by no means confined to engineering. He was a wide and careful reader of literature in many fields. He was an unassuming, but most useful member in the Church to which he belonged. His character matched his abilities; he was as patient as thorough, as cautious as courageous, and as kindly as he was firm. He was possessed of remarkable self control and, under any adverse circumstances, his serenity and poise, founded on a deep-seated courage, became evident. A great number of young engineers, many of them to-day with established reputations, profited by the instruction which he delighted to give them. This instruction was specific, thorough, untiring, and was given generously and unassumingly. He possessed the faculty of making friends, as well as admirers, of those with whom he came in contact.

---

[2] *Transactions*, Eng. Inst. of Canada, Vol. XXXII, 1919, Pt. 1, "The Quebec Bridge," p. 63; see, also, Pt. A (plates to accompanying text of Pt. I).

In erection work Mr. Porter was always interested. Derrick cars of a then new type were devised and built by him in the first years of his work in Canada, and are still in use. A recent device of his suggesting, put in place a long series of deck girder spans, completely assembled and field-painted, in a high viaduct, some of the spans running to 112 ft. in length, and this in a period of only about two hours for removing the old and placing the new span. Another of his devices erects lift bridges by moving itself vertically up the tower, and then horizontally along the bridge, erecting as it goes. The bridges and buildings with which he was connected are, of course, of great number. Various patents were granted him, and applied for by him—some of great ingenuity and value.

In the case of any active contracting bridge company it frequently happens that a noteworthy design may be made for a structure the contract for which, for various competitive reasons, is secured by another company, and the design, excellent as it may be, is not used. A striking example of this, in Mr. Porter's case, was the important Sydney Harbor Bridge, in Australia. In commenting on the Canadian Bridge Company's design on which its unsuccessful tender for this great bridge was based, Dr. J. J. C. Bradfield, Chief Engineer, made the following remarks in his report:

"The methods of stress analysis are beyond criticism, and full and accurate details of deflection, secondary stress problems, and intricate connection layouts were submitted. * * * Discussion of many intricate secondary phenomena is given in the calculations submitted with the tender. These calculations show a masterful knowledge of statical principles and practice of bridge design."

Mr. Porter was a member of the Engineering Institute of Canada, the American Railway Bridge and Building Association, the Detroit Engineering Society, and also of the Detroit Athletic Club, the Grosse Ile Golf Club, and the Detroit Automobile Club.

He was married in June, 1894, to Florence E. Murray, who, with one daughter, Mrs. Bertram S. Perham, survives him.

Mr. Porter was elected a Member of the American Society of Civil Engineers on October 1, 1913.

WILLIAM ABBOTT PRATT, M. Am. Soc. C. E.[1]

Died March 16, 1930

William Abbott Pratt was born on June 16, 1850, in Richmond, Va. His father was William A. Pratt and his mother, Eveleen (Ginevan) Pratt. On the paternal side, his grandfather, who was also William Abbott Pratt, came from England and settled in Virginia in 1832. His mother's family made their home in what is now West Virginia many years before the Revolutionary War.

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

Mr. Pratt received his early education and spent his childhood at the University of Virginia near Charlottesville in that State, his father being for many years Superintendent of the University. Mr. Pratt also attended private schools in Charlottesville, prior to his matriculation at the State University.

On leaving college he entered the employ of the Chesapeake and Ohio Railroad Company as Rodman in the Engineer Corps. Subsequent to this primary course in practical engineering experience, Mr. Pratt received the degree of Civil Engineer at the University of West Virginia. During the period between 1874 and 1876, he studied at Drexel Institute in Philadelphia, Pa., and was graduated as an Electrical Engineer.

Prior to this period, in 1869, however, Mr. Pratt was connected with the Chesapeake and Ohio Railroad Company in the Engineer Department during the location and construction of the road throughout West Virginia. In this capacity, he occupied the various positions from Rodman to Resident Engineer. In 1873, he transferred his interests to the Baltimore and Ohio Railroad Company as Resident Engineer on the construction of the Chicago Extension of that road. After an interval spent in the Maintenance-of-Way and Traffic Department, he accepted, in 1877, a position in the Engineer Department of the City of St. Louis, Mo., subsequently being made District Engineer of the District above Chouteau Avenue and between Grand Avenue and the Mississippi River.

In 1881, Mr. Pratt again entered the employ of the Baltimore and Ohio Railroad Company in charge of the location and construction of lines in Western Pennsylvania. From 1883, he was Chief Engineer on the location, construction, and operation of several short lines in West Virginia. From 1886 until 1896, he served as Division Engineer of the Philadelphia Division of the Baltimore and Ohio Railroad Company and, during this period, and until 1898, he also served as President of the Street and Sewer Directors of Wilmington, Del.

From this latter date until 1902, Mr. Pratt was Professor of Mechanical and Electrical Engineering in Delaware State College, at Newark, Del., and was also engaged in consultation practice. While in this field, he made a survey for a railroad in Alaska, by the Yukon route, from Juneau to Lake Teslin, the head of the Yukon River, extending over a distance of 150 miles. This was the first extended railroad survey in Alaska.

At the outbreak of the Spanish-American War, Professor Pratt obtained a leave of absence from Delaware State College and was commissioned Captain of Company M, First Delaware Volunteer Infantry. His service continued throughout the war and after the signing of the peace treaty, he returned to his college work.

In the summer of 1901, Captain Pratt again visited Alaska in charge of an expedition to Unalaska Island. The following year he resigned from the College Faculty to assume charge of the Baltimore and Ohio Railroad Company's interests in New York and Staten Island. Later, he resigned this position to become Chief Engineer of the New York Westchester, and Boston Railway Company.

In May, 1909, because of his failing health, Captain Pratt gave up business in New York City and retired to the family home at Waynesboro, Va. In January, 1910, he organized the Riverside Light and Power Company of Waynesboro and served as its President and Manager until January, 1920. This Power Company was purchased by the Virginia Public Service Company, and it was as Consulting Engineer for the latter that Captain Pratt served until his death which occurred from pneumonia after an illness of only a few days.

Captain Pratt had been a very active citizen of Waynesboro. He was a member of the Board of Directors of the Citizens-Waynesboro Bank and Trust Company; a Charter Member of the Waynesboro Rotary Club; a member of the City Council; Chairman of the Zoning Board of Appeals; President of the Library Board; President of the Waynesboro Welfare Alliance; and a member of the Waynesboro Literary Club. He was Senior Warden and Registrar of St. John's Protestant Episcopal Church, and a member of the Vestry, having also served as Treasurer of the Church. He was further affiliated with the Railway Signal Association and the Society of the "Virginians". Outside of his home, Captain Pratt was known to have had two great interests—his profession and St. John's Church.

He was married on February 25, 1873, in Trinity Church, in Waynesboro, to Rebecca Porterfield Kinney, of Stanton, Va. He is survived by his widow and by several nieces and nephews.

Captain Pratt was elected a Member of the American Society of Civil Engineers on July 5, 1882.

MARSHALL ROGERS PUGH, M. Am. Soc. C. E.[1]

Died November 22, 1930

Marshall Rogers Pugh was the son of John B. and Elizabeth (Fox) Pugh and came of a family which numbered among its members the Rev. J. Dickinson, the first President of Princeton University, Edward Fox, the first Secretary and Treasurer of the University of Pennsylvania, soldiers of the Revolutionary War, and a judge who served in the War of 1812. He was born on December 31, 1863, at Doylestown, Pa., about 20 miles north of Philadelphia, Pa., and followed an engineering career which took him to far corners of the United States, outside its borders, and to France where he served with distinction in the World War, and it brought him finally again to Philadelphia for many years of activity in consulting practice and in civic affairs.

Mr. Pugh attended the Towne Scientific School of the University of Pennsylvania from 1881 to 1886 receiving, in June, 1886, the degree of Bachelor of Science and in June, 1887, the degree of Mining Engineer. He stood at or near the head of his class during the entire college course. In August, 1886, his engineering work began with the Northern Pacific Coal Company in the far Northwestern United States. Until October, 1887, he was engaged as Engineer in Charge of railroad location and construction, mining engineering, mapping, construction of tipples, saw-mills, etc., at Sims, N. Dak., Timberline, Mont., and Roslyn, Wash. His work at Roslyn included the design and layout of the town.

---

[1] Memoir prepared by Stephen Harris, M. Am. Soc. C. E., and Charles A. Howland, Assoc. M. Am. Soc. C. E.

Returning, in 1887, to the University of Pennsylvania, he instructed for two years in Engineering but, in June, 1889, he journeyed again to the Northwest and was engaged for more than four years in private practice at Denver, Colo., as a member of the firm of Lang and Pugh. April of 1894 brought Mr. Pugh to Philadelphia to take up the work of Assistant Engineer in the City's Bureau of Surveys, Department of Public Works. The surveys, street layouts, sewer planning and construction, rebuilding of storm sewers, paving, and other municipal engineering work of this Bureau engaged him for more than five years.

He resigned, in 1900, from the Philadelphia Bureau of Surveys to accept the position of Construction Engineer with the organization of Charles McCall, then one of the largest contracting firms in the Eastern United States. When Mr. McCall died in 1901, Mr. Pugh entered into partnership with Mr. Hugh T. Downing for the practice of engineering and architecture. In 1903, this firm was succeeded by that of Pugh and Hubbard, of which the latter was the late Isaac Wendell Hubbard, M. Am. Soc. C. E.

During the seventeen years that Mr. Pugh was associated with Mr. Hubbard he was engaged in a variety of work in many parts of the United States and abroad. The firm designed and constructed water-works for Georgetown and Newport, in Delaware, for Manasquan and Cranberry, in New Jersey, and for Boyertown and Enola, in Pennsylvania. Sewers and sewage disposal plants were designed and constructed for seven towns in New Jersey, two towns in Delaware, four towns in Pennsylvania, for Haverford College, and for Ocean City, Md. The firm located 6 miles of trolley line in Williamsport, Pa., did the preliminary work upon 10 miles of railroad in Kentucky, and mapping and preliminary work for 45 miles of railroad in Honduras to reach mining and timber concessions. Location plans and details of 40 miles of interurban railroad in Ohio, from Wooster to Mansfield, were the firm's work. Other important trolley lines were surveyed and constructed in New Jersey, and 10 miles of narrow-gauge, interplant railroad were built for the Alan Wood, Iron and Steel Company in Pennsylvania. Among the largest of the firm's undertakings were the development of Queen Lane Manor, in Philadelphia, for the Warden Estate, and the surveying, designing, and construction, on a percentage basis, of the industrial Town of Enola, for the Pennsylvania Railroad Company.

When the United States entered the World War, Mr. Pugh offered his services and, in June, 1917, entered the army as Major, Corps of Engineers. He commanded the 1st Battalion, 21st Engineers, on the construction of light or combat railways in the Toul-St. Mihiel Sector in France and, after the

Armistice, was Post Engineer at the Bordeaux Embarkation Camp in charge of water-works, sewers, roads, and building construction. Returning to the United States, Major Pugh was retained in service by the U. S. Shipping Board, Emergency Fleet Corporation, to handle the cancellation of water contracts.

After his release from service in 1921, with the title of Lieutenant-Colonel, Engineer Reserve Corps, Colonel Pugh resumed practice as a Consulting Engineer in Philadelphia and continued his work in sewerage and sewage disposal, water supply, flood control, and highway engineering. With George L. Watson, M. Am. Soc. C. E., he founded the firm of Watson and Pugh. Some of the larger work executed by the firm included a sewerage system for Clifton, N. J., a sewage disposal plant for the Perkasie Sewage Company, sewers and sewage disposal for Neptune Township, New Jersey, storm sewers and flood control at Passaic, N. J., and also at Clifton, and sewage disposal for Ocean City, N. J.

His active interest in civic betterment engaged Colonel Pugh in many undertakings for the improvement of his city. In 1913 and 1914, he was a member of the Pennsylvania Suburban Metropolitan Planning Commission and as Chairman of its Survey Committee made studies for a comprehensive regional plan for the Philadelphia Metropolitan Area. The Commission went out of existence, but when the Regional Planning Federation of the Philadelphia Tri-State District was formed later, Colonel Pugh became a member of its Technical Advisory Committee. As a member of the Boards of the Philadelphia Housing Association and the Pennsylvania Housing and Town Planning Association, he actively promoted better housing and sanitary conditions in Philadelphia and elsewhere. When the Philadelphia Chamber of Commerce sponsored a hospital and health survey of the city, Colonel Pugh served on a Committee to co-operate in the work of sanitation. This record might be extended at great length to include activities with civic groups of the Engineers' Club of Philadelphia, the Philadelphia Conference on City Government, and many others. Where active work for civic betterment was to be done, Colonel Pugh was found in the front rank.

In 1890, Colonel Pugh was married at Doylestown, to Helen Darlington, who, with their three children, Edward John Pugh, Henry Darlington Pugh, and Elizabeth (Pugh) Welpley, survives him.

He was a member of the Protestant Episcopal Church. He was also a member of the American Waterworks Association, the Society of American Military Engineers, a Thirty-Second Degree Mason, and a member of the Engineers' Club of Philadelphia. During his active life Colonel Pugh found time to contribute articles and papers to the technical journals and to engage in the discussions of engineering societies.

No truer or more impressive account can be given of the life of Colonel Pugh than the recounting of his many achievements. By his works he is known. Thousands live in greater comfort, travel more safely and rapidly upon their business, and are secure in the enjoyment of adequate public works, because of his labors.

Mr. Pugh was elected an Associate Member of the American Society of Civil Engineers, on December 7, 1904, and a Member on November 2, 1908.

JOHN WILLIAM RAITT, M. Am. Soc. C. E.[1]

Died September 27, 1930

John William Raitt, the son of Nathan E. and May (Hall) Raitt, was born at Shawneetown, Ill., on March 21, 1880. When he was six years of age the family moved to Cincinnati, Ohio, where he lived until he reached manhood. He received his education in the Cincinnati High School and the Nelson Business College of Cincinnati, and was graduated from both institutions. Later, he continued his engineering studies through correspondence courses.

The first active work undertaken by Mr. Raitt was from 1898 to 1901, when he was employed on the construction of the Cincinnati Belt Line Railway, a part of the Cincinnati, Portsmouth and Virginia Railway (now the Cincinnati District of the Norfolk and Western Railway), successively, as Axeman and Rodman. When the Cincinnati, Portsmouth, and Virginia Railway was taken over by the Norfolk and Western Railway Company, Mr. Raitt was moved to Radford, Va., in the office of the Engineer of Branch Lines where he served as Draftsman. Following this engagement he was transferred to Kenova, W. Va., where he held a similar position from 1902 to 1905.

From 1905 to 1906 Mr. Raitt was employed in the office of the Superintendent of Construction of the Cleveland, Cincinnati, Chicago, and St. Louis Railway Company. He resigned this position and, in 1907, returned to the Norfolk and Western Railway Company, at Roanoke, Va., as Draftsman. His work in this position included all classes of drafting incidental to branch-line construction, such as mapping, right-of-way plans, track, bridge design, etc. Subsequently, Mr. Raitt was promoted to be Assistant Engineer in the office of the Engineer of Branch Lines, and, in 1918, was made Chief Draftsman in the office of the Chief Engineer at Roanoke, advancing to the position of Office Engineer in 1919.

On January 1, 1924, he was appointed Assistant Engineer at Norfolk, Va., in which position he had field charge of maintenance of way, construction, etc. This work also covered field supervision of the terminals at Lamberts Point, Va., where the Norfolk and Western Railroad Company's coal piers and warehouses are situated. Due to his experience in branch-line work, he was placed in charge of the preliminary surveys and location of the Big Sandy and Cumberland Railroad, in Buchanan County, Virginia, which road is controlled by the Norfolk and Western Railroad Company. His work here covered the preliminary surveys, location, and purchase of right of way for about 40 miles of mountain line.

In 1926, Mr. Raitt was transferred to Bluefield, W. Va., as Assistant Engineer, where he had charge of engineering on the Pocahontas Division of the Norfolk and Western Railroad. The construction of the Big Sandy and Cumberland Railroad was also under his direct charge, but, unfortunately, he did not live to complete this work.

Mr. Raitt was a man of fine character, well liked by every one with whom he came in contact, a man who was never known to have an enemy. He was a conscientious worker and was devoted to the profession which he had chosen for his life work.

---

[1] Memoir prepared by W. P. Wiltsee, M. Am. Soc. C. E.

He was married in June, 1903, to Emma Tschudi. They had three children: Nathan C., Elton W. (deceased), and John Hall Raitt. Mrs. Raitt died on February 2, 1915, and in September, 1917, Mr. Raitt was married to Mrs. Cora Carpenter Moss who, with his two sons, survives him.

Mr. Raitt was elected a Member of the American Society of Civil Engineers on May 19, 1924.

WILLIAM BOARDMAN REED, M. Am. Soc. C. E.[1]

DIED JUNE 19, 1930

William Boardman Reed was descended from several families that emigrated to the United States from England in the early part of the Seventeenth Century. He was the oldest son of Henry A. and Alice Boardman Reed, and was born in Poughkeepsie, N. Y., on May 27, 1860. His father was one of the pioneer telegraphers, a friend of Professor Samuel Finley Morse, and assisted the latter in many of his experiments, being the first person to test for faults by the galvanometer.

Mr. Reed was prepared for college at private schools and, in 1878, entered Union College at Schenectady, N. Y., from which he was graduated in 1882 with the degrees of Bachelor of Arts and Civil Engineer. On his graduation he was appointed Chief Engineer of the Lake Champlain and Moriah Railroad Company; Witherbee, Sherman and Company; and the Port Henry Iron Ore Company, the latter two companies being engaged in mining magnetite, known as Lake Champlain ore. He retained this position until 1891, and during this period was also consulted by various other mining companies, making one trip to the Island of Cuba to inspect iron ore properties there.

In 1891, Mr. Reed formed a partnership with Mr. William Gifford, of Schenectady, and carried on a general engineering practice with headquarters in that city. During this time he had charge of the construction of waterworks for the Village of Northville, N. Y. He was Chief Engineer of construction for the Cayadutta Electric Railroad, running from Fonda to Gloversville, N. Y. The Cayadutta Line was a pioneer freight and passenger electric road, and the first of its kind to carry freight. It was built over private right of way, and was operated successfully for some years, after which it was bought by and consolidated with its steam rival, the Fonda, Johnstown and Gloversville Railroad Company.

On the completion of the Cayadutta Railroad in 1894, Mr. Reed went to New York, N. Y. The Metropolitan Street Railroad Company was then in formation, and he was immediately engaged as Engineer of Maintenance. As organized by Mr. Reed, the Maintenance-of-Way Department became one of the most efficient in the country. The difficulties entailed in maintaining a conduit electric structure, in view of the extraordinary traffic and other conditions peculiar to New York City, called for engineering and executive ability of a high degree. Through handling these problems Mr. Reed became an authority on urban electric construction and maintenance. He was instrumental in designing the section of girder rail which is the standard of Greater New York and which is much used in other cities. During his connection with the street railways of New York City, Mr. Reed was widely consulted by those about to build or reconstruct electric lines in other cities.

---

[1] Memoir prepared by H. Burdett Cleveland, M. Am. Soc. C. E.

While he was acting as Maintenance Engineer for the New York City Railways, Mr. Reed was also Vice-President of the Bishop Gutta Percha Company, of which his father, Henry A. Reed was President. When the latter wished to withdraw from active work, Mr. Reed felt it necessary to retire from engineering and give his attention exclusively to this Company, which afterward became the Bishop Industries, Inc. For several years Mr. Reed continued to act as Consulting Engineer, but for about ten years before his death, he had been unable to devote any time to this work.

Mr. Reed was Vice-President of the Graduate Council of Union College at the time of his death, and had served for several years as President of the Board of Trustees of The Arietta Crane Reed Home for Convalescents, at Brewster, N. Y. He was a member of the Delta Phi Fraternity, the Engineers Club, and the New York Railroad Club, and had been President of the Union College Engineers Club since its organization.

He is survived by his widow, Sarah (Parks) Reed, to whom he was married in 1896, by Edith Harlan (Reed) Robinson, a daughter by an earlier marriage, by Marion Boardman (Reed) Hitchcock, and by his son, Henry Hazen Reed.

Mr. Reed was a thorough-going American gentleman of wide sympathies, with an unswerving loyalty to his professional colleagues, to his college, and to the many and varied interests that claimed his attention. In his own circle of friends and acquaintances the memory of his upright character, his kindly temperament, and his whole-souled devotion to any duty that lay before him, will persist.

Mr. Reed was elected an Associate Member of the American Society of Civil Engineers, on February 6, 1895, and a Member on May 1, 1901.

CLAUDE IRVIN RHODES, M. Am. Soc. C. E.[1]

DIED MAY 11, 1930

Claude Irvin Rhodes, the son of Luther Graham and Sarah (Irvin) Rhodes, was born in Clarksburg, Calif., on December 30, 1877. He received his preparatory education in the public and High Schools of Woodland, Calif., and his engineering education at the University of California, from which he was graduated with the degree of Bachelor of Science in Mining in 1902.

After extensive experience in California, Arizona, and Alaska, on problems involving surveying, stream gauging, drainage, mining, canal lining, water supply, and other work of like character, Mr. Rhodes' education, training, and experience resulted in his appointment as Engineer in Charge of the design, manufacture, and installation of 14 miles of large diameter transmission pipe line for the Monterey County Water-Works and in the location and construction of 23 miles of scenic highways on the Monterey Peninsula.

---

[1] Memoir prepared by F. M. Faude, Assoc. M. Am. Soc. C. E.

He was closely identified with the work of rebuilding San Francisco, Calif., after the great fire of 1906, and devoted more than three years to the inspection of materials for, and the installation of, sewers, public buildings, the high-pressure fire system, reservoirs, pavements, and other structures. Later, he was placed in charge of the construction and operation of the water supply and gas systems for the Panama-Pacific Exposition.

From 1918 to 1923, Mr. Rhodes was with the Hydraulic Division of the Railroad Commission of the State of California, serving with distinction as Chief Hydraulic Engineer for the last two years of this period.

In 1923, he joined the staff of the Loveland Engineers, Incorporated, of San Francisco, where his ability and wide experience were at once fully recognized and appreciated. In 1925, he became Vice-President of the organization, and at the time of his death was a Vice-President of Western Continental Utilities, Incorporated, and of The Southwest Telephone Company.

Mr. Rhodes was a member of the Commonwealth Club of California, the American Waterworks Association, and the Engineer's Club of San Francisco. He enjoyed an enviable reputation for loyalty to his friends and associates and for his ability and the painstaking care with which he approached and carried through the many and varied classes of work with which he was entrusted. He had a host of friends, and his untimely passing is mourned by all his acquaintances and associates.

He was married on January 15, 1905, to Lucy Boettiger, and is survived by his widow, a daughter, Anita Louise Rhodes, a son, Thomas E. Rhodes, and two brothers, Glenn Rhodes, of San Francisco, and Lester Rhodes, of Seattle, Wash.

Mr. Rhodes was elected a Junior of the American Society of Civil Engineers on March 5, 1907; an Associate Member on March 1, 1910; and a Member on January 17, 1927.

FRANKLIN RIFFLE, M. Am. Soc. C. E.[1]

DIED OCTOBER 29, 1929

Franklin Riffle was born on April 19, 1858, in Somerset County, Pennsylvania, the son of William and Mary (Spiecher) Riffle. The family moved West, stopping for a time at Akron, Ohio, but settling eventually on a farm near Spring Hill, Kans., where Mr. Riffle's early life was spent. At fifteen years of age he taught the district school while fitting himself for admission to the University of Kansas, from which he was graduated in 1880. Such was his scholastic record that some years later when chapters of the Sigma Xi and Phi Beta Kappa Fraternities were established at the University of Kansas, on the recommendation of his former instructors he was made a member of both these honor societies.

---

[1] Memoir prepared by the following Committee: Frank T. Oakley, E. T. Thurston, and Victor H. Poss, Members, Am. Soc. C. E., and Thomas W. Brooks, Esq., San Francisco, Calif.

Immediately after graduation Mr. Riffle found employment with the pioneering railroad development in the Northwest. Beginning in July, 1880, he occupied the position of Draftsman in the office of the Chief Engineer of the Oregon Railway and Navigation Company, and in 1881 and 1882 was an Assistant on the construction of the Columbia River Division.

On January 1, 1882, he was married to Veola Bardwell, of Lawrence, Kans.

In 1882 and 1883, he was an Assistant Engineer on the construction of the Clarks Fork Division of the Northern Pacific Railroad and, in 1884, he was employed on the construction of the Palouse Branch and also the Cascade Division of the same line. In 1885, he returned to the employ of the Oregon Railway and Navigation Company, in charge of the construction of the Pataha Branch.

In 1886 and 1887, Mr. Riffle was a Resident Engineer in charge of the construction of three branch lines of the Burlington and Missouri River Railroad in Nebraska. Again, he returned to the Oregon Railway and Navigation Company as Locating Engineer. From 1888 to 1891, inclusive, he was, successively, Superintendent of Construction and Chief Engineer of the Oregon and Washington Territory Railway, with headquarters in Walla Walla, Wash.

From 1892 to 1895, inclusive, he was associated with his brother, the late Albert S. Riffle, M. Am. Soc. C. E., in a general contracting business under the name of the Oregon Bridge Company, with headquarters in Portland, Ore. This Company was engaged chiefly in the construction of highway bridges throughout the States of Oregon and Washington, but its most noteworthy undertaking was the furnishing and laying of a 28-in. submerged cast-iron pipe across the Willamette River at Portland. The unusual nature of this project was illustrated by the fact that of sixteen leading pipe founders in the United States invited to bid, only three submitted proposals. A permanent record of the experience on this undertaking was prepared by Mr. Riffle.[2]

In 1897 he went to Victoria, B. C., Canada, and in the following year developed and managed the White Water Deep Mines. From 1899 to 1901 he was Manager of the Pipe and Mining Machinery Departments of the Wolf and Swicker Iron Works, at Portland.

In 1901 he went to San Francisco, Calif., where he accepted the management of the Pipe Department of the Dunham, Carrigan, and Hayden Company, later taking over also the Iron and Steel Department, which position he retained for twenty-two years. His identification with the commercial end of the iron and steel industry came at a time when steel was coming into general use, particularly in the soft weldable grades. Through research and practical experiment Mr. Riffle became a recognized authority on the characteristics and uses of iron and steel, and was the author of several pamphlets and monographs which were favorably received and which enjoyed a wide circulation. He was held in the highest esteem by his employers and associates and his resignation in August, 1923, was the cause of general regret.

---

[2] *Transactions*, Am. Soc. C. E., Vol. XXXIII (1895), p. 257.

Aside from his business activities, Mr. Riffle was greatly interested in the research and educational work of the Commonwealth Club of California. Also he was one of the organizers, the first Secretary, and, in 1910, the President of the San Francisco Section of the Society. He was a member of the Engineers' Club of San Francisco.

A few days after the destructive earthquake and fire in 1906, the San Francisco Section appointed a General Committee, as well as sub-committees, to study the earthquake effects on various classes of structures, the reports of which were published by the Society.[3] Franklin Riffle was a member of the General Committee and also of the Committee on the Effect of the Earthquake on Water-Works Structures. The lively interest he took in this study is evidenced by his discussion of the report on "Damage to Buildings".[4]

Mr. Riffle definitely retired from business in the summer of 1924 and in August of that year departed on a tour of Europe. After a year of travel in the British Isles and on the Continent, he took up his residence in St. Cyr, France, whence he made numerous tours through France, Spain, Italy, and other continental countries, often going afoot, as walking had always been with him a favorite means of recreation. A distance of from 16 to 20 miles over hill and dale was an easy day's tramp for him.

In June, 1929, an ailment of long duration became acute, and he entered the American Hospital at Paris, France, for treatment. He left the hospital on September 27 in good spirits and supposedly on the road to recovery, although continuing precautionary daily treatments. However, suffering a relapse after ten days, he returned to the hospital where he grew rapidly worse and passed away three weeks later, on October 29, 1929.

Both professionally and ethically Franklin Riffle represented the highest standards of the Engineering Profession. All those associated with him respected his high ideals. Young men who were so fortunate as to spend the early years of their professional training under his guidance were enthusiastic admirers of his character and his thorough and systematic business methods, and were always loud in praise of his kindness as manifested toward them. They looked up to him with pride as a leading member of the profession.

He was a man of simple habits and practiced moderation in all things. A logical thinker with an unusually well-balanced mentality and even temper, he was always cheerful and optimistic; irritability and needless worry were foreign to his nature. In spirit and appearance he was young for his years, keeping himself informed on the important events of the day, which he discussed in the faultless English of which he was a master. These and his other fine qualities made him a delightful and lovable companion. Contact with him was ever satisfying and beneficial. Few men have had the universal esteem and respect of their fellows that Franklin Riffle enjoyed.

As a personality, once seen he was not soon forgotten. His tall, well-knit figure, high-held chin, kindly, deep-set eyes beneath dark shaggy brows looking straight ahead whether moving briskly with easy stride and preoccupied mien along the street or mingling socially with his fellows, made him always an impressive person. Although of a cordial and friendly disposition, he was on excellent terms with himself and frequently elected to walk alone. He loved the great outdoors and during his residence in California, he maintained a cabin high in the foothills overlooking the Napa Valley where he and Mrs. Riffle, occasionally with friends, were wont to spend their week-ends and short vacations.

---

[3] *Transactions*, Am. Soc. C. E., Vol. LIX (December, 1907), p. 208.

[4] *Loc. cit.*, p. 266.

Mentally, as physically, he seemed to hold aloof from the petty, inconsequential things about him and to dwell serenely and thoughtfully in an amosphere all his own, to which he welcomed his friends with all the kindliness, consideration, and cordiality of the cultured gentleman.

Mr. Riffle was elected a Member of the American Society of Civil Engineers on November 7, 1898.

EDWARD AUSTIN RIX, M. Am. Soc. C. E.[1]

DIED JANUARY 8, 1930

Edward Austin Rix, the second son of Judge Alfred Rix and Chastina (Walbridge) Rix, was born on January 29, 1855, in a house which then stood at 640 Market Street, opposite the Palace Hotel, in San Francisco, Calif. His parents were of long established New England ancestry, his father having come around Cape Horn to San Francisco, in 1849, his mother arriving not long afterward. Mrs. Rix died when the boy was only a year old and his upbringing was under the care of near relatives.

Mr. Rix was educated in the public schools of San Francisco, and in September, 1873, entered the University of California, which he attended for four consecutive years. His studies were in the College of Mechanics, and he was a member of the first class to be graduated from that Department. The graduation exercises were held in the open air under the famous oaks in Faculty Glade, June 6, 1877. He received the degree of Bachelor of Philosophy; he also was given a commission as Captain in the Cadet Corps.

Throughout his college life he was a member of the Zeta Psi and Sigma Chi Fraternities, was remarkably active in all college affairs, universally regarded as a "good sport", and highly popular with all his fellows. He belonged to the Durant Rhetorical Society, and was a fluent and clear speaker. He was also Editor of the *Berkeleyan,* one of the college publications.

Practically from the moment of his graduation from college Mr. Rix was enthusiastically interested in the design and manufacture of compressed-air machinery. In the fifty-two years of his connection with this work he rose to be one of the acknowledged leaders in the industry. If success may be correctly defined as the realization of one's ideals, he was indisputably a successful man.

---

[1] Memoir prepared by Harold F. Gray, M. Am. Soc. C. E.

His labor was unremitting in his chosen field, and he had a deep interest in every scientific subject in any way allied with it.

His achievements make a long list. He was one of the first to recognize the use to which compressed air could be put in machinery, and devoted his engineering knowledge to its development. He held many patents on his inventions, and kept these patents from becoming obsolete by adding valuable improvements prior to the time of expiration. Among these are: The compressed air automatic hammer; the first direct-connected tangential water wheel on an air compressor; the air compressors for the dynamite gun plant at Fort Riley, Kansas (1896); the first variable-volume air compressor; and the first supercharger air compressor.

Mr. Rix was awarded the Gold Medal of the Pacific Coast Gas Association for his treatises on gas transmission and compression. He built the first liquid air machine on the Pacific Coast, and read the first paper in California on liquid air, at Stanford University, in 1895. He published the first handbooks on compressed air engineering, and on the use of air-hammer drills in mining. He was President of the Rix Companies, Incorporated, of San Francisco and Los Angeles, Calif., manufacturers of an extensive line of compressed-air machinery.

Mr. Rix had a high artistic sense, and some of the ingenious devices made with his own gifted hands are memorials to his fine taste and indefatigable industry. His fertile mind worked along individual lines, betokening an exceptional intelligence.

He had his own philosophy of life, and it was genial, kindly, and sympathetic. To his friends he was devotedly loyal, and by them he was deeply loved in return. His capacity for friendship was limitless, and was swayed by no motives apart from the worthiness of that friendship.

Mr. Rix was one of the founders of the famous Olympic Club of San Francisco; he was also a member of the Claremont Country Club, of Oakland, Calif. He was a member of the American Society of Mechanical Engineers, the American Association for the Advancement of Science, and many other scientific organizations.

Mr. Rix was married twice. His first wife was Kate Elizabeth Kittredge, of San Francisco, to whom he was married on May 2, 1878. They had four children, Genevieve (Mrs. Burrows), Chastina (Mrs. Sterling), Austin Julian, and Harold (deceased). His second wife, to whom he was married on September 24, 1913, was Gail Wheeler Shipman, of Emerson, Iowa. They had one child, Elizabeth.

He was in failing health for some years before his death, from serious heart trouble which impeded his activities, but did not daunt his interest in his life occupation. He was seized with a cerebral hemorrhage early one morning in January, 1930, and died on January 8, a few minutes after his wife, summoned from a visit to her parents in Iowa, reached his bedside. The funeral was held on Saturday, January 11, and his remains were placed in the mausoleum of Mountain View Cemetery, in Piedmont, Calif.

Mr. Rix was elected a Member of the American Society of Civil Engineers on April 7, 1897.

THOMAS RODD, M. Am. Soc. C. E.[1]

DIED AUGUST 3, 1929

Thomas Rodd was born in London, England on June 13, 1849, the son of Horatio and Anne (Theobald) Rodd. His father was a bookseller, distinguished for his intimate knowledge of books and portraits; and his grandfather, Thomas Rodd (1763-1822), was an eminent member of that profession, as well as an author and scholar of note. His family emigrated from England in 1856, settling in Philadelphia, Pa.

In October, 1862, at the age of thirteen, Thomas Rodd entered the United States Navy, and served continuously until the close of the Civil War. He rose to Ship's Writer, Acting Mate, and Captain's Clerk; his last service was on board the United States Steamer *Galena,* under Capt. Francis Wells, who recommended his appointment to the United States Naval Academy. He became a Cadet Midshipman at Annapolis in 1865, and passed all the examinations there, except the final one, which occurred shortly after his resignation. In the course of his studies, he had resolved upon an engineering career, and, finding that graduation would involve the delay of an extended cruise, he resigned in February, 1869, without waiting for the final examination and graduation. Soon afterward he was employed by Jesse Lightfoot, Engineer and Surveyor of Germantown, Pa., as a Rodman. In the same year, he obtained a position with the City of Philadelphia, under Strickland Kneass, Chief Engineer and Surveyor, in which he remained until 1872.

Mr. Rodd entered the service of the Pennsylvania Railroad Company on May 20, 1872, as a Rodman and, subsequently acted in the following capacities: From March 1, 1873, to July 1, 1877, Assistant Engineer on construction work; from July 1, 1877, until July 1, 1889, Principal Assistant Engineer; from July 1, 1889, until January 1, 1901, Chief Engineer of the Pennsylvania Company; and from January 1, 1901, until July 1, 1918, Chief Engineer of the Pennsylvania Lines West of Pittsburgh, Pa. From July 1, 1918, until the time of his retirement, June 30, 1919, during the period of Government control, Mr. Rodd was Corporate Engineer of the Pennsylvania Lines West of Pittsburgh. From 1905 to January, 1927, he was also Consulting Engineer of the Pittsburgh, Fort Wayne, and Chicago Railway Company. From November 24, 1913, until May 1, 1919, he served as Chief Engineer of the Chicago Union Station Company, which erected the new and magnificent terminal at Chicago, Ill., under his supervision. He was largely responsible for the design and plans adopted for this station. From May 1, 1919, until December 31, 1925, he was Consulting Engineer for the Chicago Union Station Company. In 1887, as Engineer and Architect, he designed and supervised the construction of the Union Station for the Indianapolis Union Railway Company, at Indianapolis, Ind.

Mr. Rodd was prominently connected with the rebuilding of the Pennsylvania Railroad after the historic flood of Johnstown, Pa., in 1889. In addition to his work with the Pennsylvania Railroad Company, he had a private practice as Engineer and Architect and designed and constructed many large structures, among which were the following: The Westinghouse Electric and Manufacturing Company's Plant and the Westinghouse Machine Company's Works, at East Pittsburgh, Pa.; the Union Switch and Signal Company's Works at Swissvale, Pa.; and the plant of the British Westinghouse Electric and Manufacturing Company, at Trafford Park, Manchester, England.

---

[1] Memoir prepared by Robert Trimble, Esq., Pittsburgh, Pa.

In 1884 and 1885, Mr. Rodd made a comprehensive investigation and report on the ore and coal-handling facilities of the Pennsylvania Railroad Company at the various ports on Lake Erie. At this time, the machinery in use was extremely crude with high labor usage; and, as a result of his recommendations, modern machinery, such as car dumpers and Brown hoists, was installed, which resulted in quickening and cheapening the handling of coal and ore. He also had an active part in the adoption of hopper-bottom cars for use with these materials, which resulted in decreased cost and increased speed in the handling of them.

From the time that Mr. Rodd became Principal Asssistant Engineer, he took an active interest in the designing and building of bridges and was a leader in foreseeing the future development of heavier loads; as a consequence, the bridges built under his administration on the Pennsylvania Lines West of Pittsburgh were decidedly in advance, in carrying capacity, of those of other lines. In this work he was assisted by such engineers as the late William Scherzer, M. Am. Soc. C. E., the inventor of the Scherzer lift bridge, and August Ziesing, M. Am. Soc. C. E., who, later, became President of the American Bridge Company.

At the time Mr. Rodd became connected with the Pennsylvania Company, the maintenance of the railroad was in charge of road-masters, men who received their training as trackmen. He was active in the replacement of these trackmen by men of engineering training, a practice which is general to-day on the System. He also interested himself in unified standards for railroad maintenance and construction.

Mr. Rodd was a Charter Member of the Engineers' Society of Western Pennsylvania, which was organized in 1880. He was a member of the first Board of Directors, and Vice-President in 1881, and, again, in 1882. He also served on several of its committees and took an active interest in the welfare of this Society. He was a Charter Member of the American Railway Engineering Association, organized in 1899, and served as a member of the Committee that drafted its Constitution and By-Laws. He was a Director in the Association from 1899 to 1903.

He took an active interest in the work of the Society. He was a member of the Committee on Proper Relations to Each Other, of the Sections of Railway Wheels and Rails, as well as a member of the Committee on Rail Sections; and, after the adoption of the Am. Soc. C. E. Standard Sections for Rails, he succeeded in having them adopted as standard on the Pennsylvania Lines West of Pittsburgh.

On October 23, 1879, Mr. Rodd was married at Pittsburgh, to Mary Watson Herron. Mrs. Rodd died in 1917. He is survived by two sons, William H. and Thomas Rodd, Jr., and a daughter, Mrs. Mary Herron Kelsey.

He died at his home in Pittsburgh, on Saturday, August 3, 1929, of heart trouble after a short illness.

Mr. Rodd was elected a Member of the American Society of Civil Engineers on June 5, 1878.

HANS HENRIK RODE, M. Am. Soc. C. E.[1]

Died July 18, 1930

Hans Henrik Rode was born in the small mining town of Roeros, Norway, on January 29, 1885, where his father was a lawyer, and, later, a judge. He received his technical education in Germany and was graduated in 1908 as a Civil Engineer from the Polytechnical College of Hannover. His first position was with the firm of Louis Eilers, in Hannover, as a Designer in its Bridge Department, acting at the same time as Assistant in Statics and Steel Structures at the Polytechnical College.

In 1909, Mr. Rode followed the call of the New World and was for three years Assistant to Gustav Lindenthal, Hon. M. Am. Soc. C. E., Consulting Engineer, New York, N. Y. He was engaged as a Supervisor of Design with the McClintic-Marshall Company, Pittsburgh, Pa., on the design of the New York Connecting Railroad, including the Hell Gate Bridge, and on the design and field inspection of the Kentucky River High Bridge. He also took active part in developing the plans of Mr. Lindenthal's unique, competitive design for the Quebec Bridge, a braced, three-hinged suspension bridge with a center span of 1 758 ft.

In 1913, Mr. Rode was again back in Germany as Chief Engineer for the Steel Bridge Department of Steffens and Noelle, Berlin-Tempelhof. He retained this position until the fall of 1914, when he was called by the Norwegian Government to be Professor of Technical Mechanics at the Polytechnical College in Trondhjem (Nidaros), and was allotted two years to "round out" his theoretical studies. He spent that period in Germany on a Government scholarship and received his Doctor's Degree in 1916 from the Polytechnical College in Hannover on a thesis entitled, "Beitrag zur Theorie der Knickerscheinungen". In the fall of this same year he was finally appointed to his Professorship, which he held until his death on July 18, 1930.

From 1924 to 1926, Professor Rode was in the United States and again working with his former chief, Mr. Lindenthal, this time as Resident Engineer in charge of the Burnside, Ross Island, and Sellwood Bridges, which were built for Multnomah County, in Portland, Ore. Of these, the Burnside Bridge has a large bascule span; the Ross Island Bridge is characterized by its graceful appearance and its ingenious cantilever scheme; and the Sellwood Bridge has the distinction of having continuous girders over four openings.

In addition to the thesis for his doctorate degree, Professor Rode had written a textbook on Mechanics and different articles for Norwegian technical magazines. As a professor, he was very popular with the students, not only on account of his interesting lectures, but also because of the vivid interest which he took in everything concerning the students. Another reason why Professor Rode appealed so greatly to his students was because of his happy blending of a keen theoretical insight and practical executive ability. Mr. Lindenthal regarded Professor Rode highly for his unusual theoretical and executive ability.

---

[1] Memoir prepared by Gustav Lindenthal, Hon. M. Am. Soc. C. E.

It was therefore a valuable life, with an excellent professional record and a still more promising future, that was abruptly cut off by Professor Rode's sudden death on July 18, 1930. He and his wife were on their way home to Norway after an automobile trip to Italy; when motoring through the Tyrol their car got out of control on a slippery road and toppled into a river, where it overturned, with the tragic result that both were drowned. Thus, by a blind accident, in the prime of his manhood, an unusually able engineer and a fine man was lost to the profession.

Professor Rode was elected a Member of the American Society of Civil Engineers on August 31, 1925. He was also a Member of the Norwegian Society of Engineers, Oslo, Norway.

JOHN HENRY ROSTOCK, M. Am. Soc. C. E.[1]

DIED MARCH 18, 1929

John Henry Rostock, the son of John and Mary Rostock, was born on August 20, 1858, at Milwaukee, Wis.

On August 16, 1878, six days before Mr. Rostock was twenty years of age, he enlisted in the United States Army and was assigned to Troop K, Eighth U. S. Cavalry, stationed at Fort Clark, Texas. In December, 1880, he was transferred to Department Headquarters at San Antonio, Tex., where he served as Clerk to the Department Engineer, Captain (now Colonel) W. R. Livermore, U. S. Corps of Engineers. Mr. Rostock was promoted to be Topographical Assistant in June, 1881, serving in the office at Headquarters and in the field, until May 17, 1886.

In 1880, while on a surveying expedition into that part of Texas lying west of the Pecos River, toward the Rio Grande on the west, and into Southern New Mexico on the north, under the command of Captain Livermore, he was sent to select sites for new military posts. While he was surveying in the territory, hostile Indians appeared, and Captain Livermore, joining his troops with those under the command of General Griersom, prepared to drive them out. Mr. Rostock was ordered to take a position in front of the troops as a scout at a distance of 100 yd. He immediately took the lead, the troops following in a single file through a very narrow, winding, and rockbound pass which led southward from Rattlesnake Spring. The Indians were encountered in the Diablo Mountains, but finally escaped into Mexico. For his valor and courage, Mr. Rostock was awarded the Indian Campaign Badge.

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

Several of his engagements with the Engineer Department, U. S. Army, during a subsequent period were as follows: 1886 and 1887, Chief Clerk, on the improvement of the Missouri River from its mouth to Sioux City, Iowa; 1887 to 1895, Assistant Engineer, river and harbor improvements, Newport, R. I., District; 1896, employed in the office of Commissioner of Highways of Rhode Island; and 1896 to 1898, Inspector of Material used in the construction of lighthouses of the iron tubular type at Spring Point, Portland Harbor, Me., and Butler's Flats, New Bedford Harbor, Mass.

From 1898 to 1906, Mr. Rostock was in charge of the construction of fortifications at Fort Warren, Fort Strong, and Fort Andrew, in Boston Harbor. His engineering work during this period included the emplacements for a number of guns of large and small caliber and designs for a drainage system for the perpetual removal of water from pockets of quicksand; he successfully built part of a mortar battery on this underdrained quicksand, as a foundation. Later, he was in charge of cement tests to determine the comparative value of various sands and granite crusher dust in cement mortars. He prepared specifications for the purchase of practically all materials and supplies at an expenditure of $500 000.

In 1906, he became Principal Assistant Engineer, New York District No. 1, Col. W. M. Black, Corps of Engineers, U. S. Army (now Brigadier-General, U. S. Army (*Retired*)), M. Am. Soc. C. E., in charge. At his death, Mr. Rostock was Senior Engineer of the New York First District and his last services were rendered in that District and also on the Board of United States Civil Service Examiners.

In all, Mr. Rostock served in the War Department for fifty-one years. Untiring and energetic, he labored incessantly in the performance of distinctly important work affecting the improvement of harbors of the Eastern Coast from Portland, Me., and Newport, R. I., to New York, N. Y., toward the development of which he contributed materially and prominently.

He was a member of the Society of American Military Engineers and the International Navigation Congress. He was a member of Ashler Chapter, A. F. and A. M., of Rockport, Mass.

In 1898, during the conflict with Spain, and in the recent World War, when Mr. Rostock volunteered for active service, his superior officer, as well as the authorities in Washington, advised him that his services in the home district were too important and too greatly needed for him to be spared in any other capacity. However, he was one of the founders of the Home Guard in Mamaroneck, N. Y., where he resided during the war, and served in that way.

He was married in Boston, Mass., on August 11, 1894, to Mary Virginia Hill, a daughter of Caspar Hill, Sculptor, of Newport. He passed away on March 18, 1929, at Mamaroneck. He is survived by his widow, and one daughter, Mrs. Lawrence Waterbury. At his burial he was accorded military honors. In addition to the twenty-five officers of the U. S. Engineers Corps who attended, a firing squad of men who had known and loved him acted as escort to the Greenwood Union Cemetery where he was laid to rest, a salute being fired and taps sounded as his flag-draped coffin was prepared for final burial.

The following tribute was paid him by the Board of United States Civil Service Examiners at the time of his death:

"April 12, 1929.

"The Local Board of U. S. Civil Service Examiners, Engineer Department at Large, New York City, records, with sorrow, the death, in his 71st year, of its fellow member

JOHN H. ROSTOCK

which occurred at Mamaroneck, N. Y., on March 18, 1929.

"For over half a century he rendered distinguished service in the War Department of the United States Government, as a soldier in the 8th Cavalry and in the civilian service as an Engineer, his last service having been rendered in the First New York Engineer District. For this, as an accomplished Engineer and a member of the American Society of Civil Engineers, he was eminently fitted. Untiring and energetic he labored incessantly in the performance of distinctively important work affecting the improvement of the harbors of Newport and New York to the development of which he contributed materially and prominently.

"During the many years up to his demise as a member of the Local Civil Service Board his experience in this capacity proved of constant value to his associates on that Board.

"His character was above reproach and in his dealing with his fellow men he was upright and just.

"We deplore the loss of this conscientious Christian gentleman and exemplary Husband and Father.

"To his surviving devoted family, to whom he leaves a proud heritage, we tender our heartfelt sympathy in their great bereavement."

Mr. Rostock was elected a Member of the American Society of Civil Engineers on May 3, 1910.

HARRY HARWOOD ROUSSEAU, M. Am. Soc. C. E.[1]

DIED JULY 24, 1930

Harry Harwood Rousseau, the son of Jeanette (Parker) and William White Rousseau, was born in Troy, N. Y., on April 19, 1870. Valedictorian of the Class of 1887 of the Troy High School, he entered the Rensselaer Polytechnic Institute in the same year. He worked during vacations (as Inspector, Troy Public Improvement Commission, in 1889 and 1890), and was graduated with the degree of Civil Engineer in 1891. Joining the Rensselaer Society of Engineers early in his student days he found time, among other activities, to be one of the staff of *The Polytechnic,* the monthly publication of the Institute, and was Editor-in-Chief during the school year, 1890-1891.

Mr. Rousseau's first work after graduation was with Charles F. Stowell, M. Am. Soc. C. E., Structural Engineer, Albany, N. Y., in 1891 and 1892; in 1893, he served as Assistant Engineer with the Brooklyn Elevated Railroad Company; and from 1894 to 1897, inclusive, with the Pittsburgh Bridge Company, as Assistant Superintendent, in the Structural Department (1894); in the Design and Estimating Department (1895-1896); and as Principal Assistant Engineer (1897).

---

[1] Memoir prepared by F. T. Chambers, Rear-Admiral, C. E. C., U. S. N., M. Am. Soc. C. E.

The following year he took a competitive examination for entry into the Civil Engineer Corps of the Navy, which, at that time, required a degree from a recognized technical institution and five years of active practice, two of which must have been in responsible charge. As he was first in merit, Mr. Rousseau was appointed, with the rank of Lieutenant, Junior Grade, on October 12, 1898. Of his first eight years in the Navy, he spent about four at Navy Yards, engaged upon the maintenance and construction of various public works of the Naval Establishment and the remainder at the Bureau of Yards and Docks, the headquarters for Naval public works activities. He was stationed at the Mare Island Navy Yard, in California, for several years and was there on November 26, 1906, when his former chief, the late Rear-Admiral Mordecai T. Endicott, C. E. C., U. S. N., Past-President, Am. Soc. C. E., reached the retiring age, and Lieutenant Rousseau was appointed by President Roosevelt, at the age of 36 years, as Chief of the Bureau of Yards and Docks, with the rank of Rear-Admiral for a term of four years from January 6, 1907.

He was not, however, to serve any considerable part of this term. Several Panama Canal Commissions had been appointed, all composed mainly if not entirely of elderly men. President Roosevelt determined to have younger engineers and to establish them at Panama. It was also decided that the canal construction should go forward under the War Department and its Corps of Engineers. The late Maj.-Gen. George W. Goethals, U. S. A. (*Retired*), M. Am. Soc. C. E. (then Lieutenant-Colonel, Corps of Engineers, U. S. A.), was selected to head the Commission and Rear-Admiral Rousseau, having served less than three months with the highest regular rank then in the Navy, reverted to the rank of Lieutenant and became a Panama Canal Commissioner with Colonel Goethals. He did of course receive the higher compensation allowed by Congress for Canal Commissioners. In addition to the duties of Commissioner, each member was assigned as head of a department under Colonel Goethals as Chief Engineer. Commissioner Rousseau became head of the Departments of Building Construction, Motive Power and Machinery, and Municipal Engineering, with more than 10 000 employees under him. Upon the general re-organization in 1908, he was placed in charge of the Second Division of the Chief Engineer's Office, with the title of Assistant to the Chief Engineer, and this position he held until the Isthmian Canal Commission was terminated by Congress, April 1, 1914. Colonel Goethals then became the first Governor of the Panama Canal and Commissioner Rousseau became Engineer of Terminal Construction.

He seems to have acquired an early habit of taking on additional duties. In 1911, in addition to his other work, he was given charge of the design and construction of the canal terminals, including the dry docks, ship repair shops, piers, coaling plants, fuel oil plants, breakwaters, floating cranes, etc. Those members of the Society who have inspected the canal activities will realize the great extent of this assignment. It was to complete these ancillary works of the canal establishment that Lieutenant-Commander

Rousseau remained on the Isthmus after the Commission, as such, was no more. The only exception Colonel Goethals ever made to his announced principle, as a matter of organization, that "no one was indispensable to the canal work", was when, in reply to Secretary of the Navy Meyer's cable asking him to release Lieutenant Commander Rousseau from the canal work so that he could re-appoint him Chief of the Bureau of Yards and Docks, he cabled that he regretted that he could not do so as "Rousseau was indispensable to the canal work." As a result, he remained at Panama until the terminals were successfully completed within the estimated cost and the estimated time. The cost was more than $20 000 000.

By an Act approved March 4, 1915, Congress extended its thanks to Lieutenant Commander Rousseau as a member of the Isthmian Canal Commission for having "rendered distinguished service in constructing the Panama Canal", and authorized the President to advance him to the grade of Rear Admiral. For fifteen years, to the day of his death, he served on the active list in this rank. The terms of the Congressional Act would also have permitted him to retire as others of the Canal Organization in like case did and to reap in civil life the financial rewards which, without doubt, could have been his. Some have asked why he did not do this and others have known without asking that it was a characteristic of the man. He felt that his Government had treated him well; that he owed it his active assistance so long as his efforts were found of particular value. That his efforts were highly appreciated is conclusively proved by the record of his subsequent career.

Returning from Panama, Admiral Rousseau was appointed a member of the Commission on Navy Yards and Naval Stations which existed over a considerable period of years and which surveyed the three coasts of the Continental United States for navy yards, naval bases, submarine bases, and aviation bases. Particularly on the Pacific Coast, and most notably at San Francisco Bay, an exhaustive study was made for a main West Coast base. This and other reports were published as Congressional documents. Meanwhile, the World War engaged the attention of the Nation, and ships for the transport of both men and material were required in large numbers over and above the existing supply. The United States Shipping Board organized the Emergency Fleet Corporation to design and build the needed ships. Before such ships could be built in requisite numbers, however, additional shipyards must be designed and built. Admiral Rousseau became Assistant General Manager and Head of the Shipyards Plant Division of the Emergency Fleet Corporation.

Besides designing and building new shipyards and additions to existing commercial yards, the Shipyards Plant Division allocated and designed many marine railways and dry docks to care for the ship-building program, and all this work was under Admiral Rousseau's supervision. In addition to his duties with the Emergency Fleet Corporation, Admiral Rousseau served as Associate Director of the United States Housing Corporation, in charge of Naval housing matters; Project Manager in charge of the expenditure of Naval funds for improvements to plants of shipyards having contracts for Naval vessels; and as a member of the Government Munitions Board. For his efforts he was awarded the Navy Cross "for exceptionally meritorious service in a duty of great responsibility." Admiral Rousseau was also a member of the Port Facilities Commission of the United States Shipping Board, serving part of the time as its Vice-Chairman.

After the war, he continued his Navy Yard Commission duties, to which were added many others. He was constantly serving on Boards for various purposes, such as President of numerous Naval Examining Boards for the appointment and promotion of officers in the Corps of Civil Engineers, U. S. Navy; Alternate Navy Member of the Interdepartmental Board on Contracts and Adjustments, under the Bureau of the Budget, from 1922; Senior Member of the Board appointed by the Secretary of the Navy to consider claims of land owners arising from the commandeering of land for the Naval Operating Base at Hampton Roads, Va.; and Senior Member of numerous Boards to report on the settlement of Navy war contracts with the Bureau of Yards and Docks. Since 1923, Admiral Rousseau had served as Chairman of the Metals Committee, Federal Specification Board, under the United States Bureau of Standards. For three years he was Government Receiver in the oil litigation over Naval Petroleum Reserve No. 1, known as the Elk Hills suit, receiving the commendation of the Court for his assistance in this difficult matter. His work in connection with the Naval oil reserves was extended and, for three years prior to his death, he was Director of the Naval Petroleum and Oil Shale Reserves. Meanwhile, in addition to his other duties, he was made Chief Co-Ordinator of the Bureau of the Budget to administer, under the Director of the Budget, all Federal co-ordinating agencies, a position of great responsibility, requiring close detailed application. While recognizing to the full Admiral Rousseau's tremendous abilities and capacity for work, his intimates nevertheless believe his close attention to Budget matters was contributory to his final collapse.

Admiral Rousseau, since Panama Canal Commission days, had been a Director of the Government-owned Panama Railroad Company which operates the railroad in the Canal Zone, and the Panama Railroad Steamship Company. He died at sea on the S. S. *Cristobal,* on his way to the Canal Zone to make an inspection.

Admiral Rousseau had delivered addresses and presented papers before various technical societies, covering miscellaneous engineering subjects, port development, oil shales, and the Panama Canal. In 1923, he delivered the principal graduating address at Rensselaer Polytechnic Institute.

A first-rate engineer, an able executive, Admiral Rousseau's general demeanor was one of unusual reserve. His was, however, a kindly nature. He was ever ready to help. He carried the respect, the high regard of those who knew him in his work. Socially, he was quite responsive. With a keen sense of humor, among his friends he was both an enjoyer and a lender of gayety to the party.

He was a member of the Rensselaer Society of Engineers and of the Engineers Society of Western Pennsylvania, the American Association of Port Authorities, and the Permanent International Association of Navigation Congresses. He was also a member of the Army and Navy, the Metropolitan, and the Chevy Chase Clubs, of Washington, D. C., and of the New York Yacht Club. A staunch Churchman all his life, he was, at the time of his death, a Vestryman of St. Thomas Protestant Episcopal Church, Washington, from which the funeral was held. He was buried in Arlington National Cemetery with full military honors.

Admiral Rousseau found his bride at Panama. He was married to Gladys Fargo Squires, the daughter of the American Minister to Panama, in 1908. She, with three sons, Henry H., William P., and Bard S., survives him. He also leaves behind him a brother, W. W. Rousseau, Assoc. M. Am. Soc. C. E., of the Rensselaer Polytechnic Institute, at Troy.

Admiral Rousseau was elected a Junior of the American Society of Civil Engineers on June 6, 1893; an Associate Member on April 4, 1900; and a Member on January 18, 1916.

RICHARD LORD RUSSELL, M. Am. Soc. C. E.[1]

Died March 16, 1930

Richard Lord Russell was born in Brooklyn, N. Y., on February 27, 1874. Both his father, William Warren Russell, and his mother, Margaret Butler (Hartt) Russell, were descended from early Colonial New England stock. He received his technical training at the Polytechnic Institute of Brooklyn, earning the degree of Bachelor of Science in 1893 and that of Civil Engineer in 1897. He took a post-graduate course from September, 1893, to February, 1894.

From February to June, 1894, Mr. Russell served as Instructor in Civil Engineering at his Alma Mater. From June, 1894, until September, 1896, as Leveler for the Department of Water Supply, Brooklyn, he had charge of a transit party locating a new steel pipe line, and surveyed locations for new driven-well stations at Freeport, Merrick, and Massapequa, N. Y. In September, 1896, he returned to the Polytechnic Institute as Assistant Professor of Civil Engineering, which position he held until September, 1897.

From February to December, 1898, as Resident Engineer of the Nassau Electric Railroad, Mr. Russell superintended the erection of an addition to the Third Avenue Power Station. From January, 1899, to July, 1902, he was with the Brooklyn Heights Railroad Company, beginning as Assistant Engineer. In April, 1899, he was promoted to Assistant to the Chief Engineer and in January, 1900, became Assistant Engineer on Maintenance of Way.

In July, 1902, as Resident Engineer for the Flatbush Avenue Terminal and the Carlton and Sixth Avenue Freight Yards on the Atlantic Avenue Improvement, for the Long Island Railroad Company, he made topographical surveys and designed the track layout, masonry, drainage system, and other items for a 16-track underground terminal station. He also made the designs for three blocks of freight yards and had active charge of the construction of all these projects.

---

[1] Memoir prepared by Rex B. Mesny, Esq., Phoenix, Ariz.

From February to August, 1905, Mr. Russell served as Principal Assistant Engineer on the Brighton Beach Improvement, in charge of the construction of Section 1 and the surveys and plans for Sections 2 and 3. During this time he built the depressed section of the Brighton Beach Railroad under traffic.

In August, 1905, he became Chief Engineer for Charles Cranford, General Contractors, and, in 1908, he added to this the duties of the same office for the Borough Development Company of Brooklyn. He held both these positions until 1913, when he became Vice-President of the Sanborn-Russell Construction Company. Mr. Russell was a co-patentee of the Sanborn-Russell High Temperature Incinerator. The following year he became a partner in the firm of Harrison and Russell.

The next two years, from 1915 to 1917, Mr. Russell spent in the capacity of Chief Engineer of the Eastern Contracting Company of Brooklyn; and the following two, from 1917 to 1919, as Consulting Engineer of the Uvalde Asphalt Company of America.

From 1919 to 1922, he was with The Foundation Company. During this time he had charge of the construction of a 936-ft. mine shaft for the Miami Copper Company, at Miami, Ariz. Subsequently, he was Works Manager on the construction of extensions to the Standard Oil Refinery at El Segundo, Calif., and, later, District Manager for the Company in the Los Angeles territory. In 1923, he went with the Austin Company as Sales Engineer.

Mr. Russell was greatly interested in construction engineering, and devoted considerable time to the work of the Construction Division of the Society. During the last year of his life he did a great amount of research work on sound-proof construction for the talking films. He had an open, genial nature, and was highly respected and loved by all the construction men who came in contact with him. Being an inveterate reader, he collected an extensive library, and was a conversationalist of rare charm. He was the author of "Contractors' Cost Keeping" and "Sinking and Concreting 936 Foot Mine Shaft."

The radio was one of his hobbies, and another was the study of Nature, in which interest he was joined by his wife. He was deeply devoted to his family, and enjoyed participating with them in many sports, the chief of which was yachting. In fact, his boat, *Patricia*, won many cups in the skimmer class on the Western Coast.

Mr. Russell was married in Brooklyn, on October 10, 1900, to Ruth Eloise Maddren, who, with a son, Richard, and a daughter, Dorothy, survives him.

He was a member of the Corporation of his Alma Mater, the Polytechnic Institute of Brooklyn, from 1907 to 1918, having served as Secretary of the Corporation from 1913 to 1918. He was also a member of the Protestant Episcopal Church; the Republican Party; the Engineers (Brooklyn), University, Jonathan, and City (Los Angeles) Clubs; the American Society of Engineering Contractors; the American Association of Engineers; and the National Geographic Society. He was the first Commodore of the Alamitos Bay Yacht Club.

Mr. Russell was elected a Junior of the American Society of Civil Engineers on February 5, 1895, an Associate Member on April 3, 1901, and a Member on September 4, 1906.

GEORGE FREDERICK SAMUEL, M. Am. Soc. C. E.[1]

Died November 5, 1930

George Frederick Samuel was born in New York, N. Y., on May 8, 1859. He was graduated from the University of Michigan with the degree of Bachelor of Science in Civil Engineering in 1885.

In this same year, Mr. Samuel accepted a position as Assistant Engineer for the Town of Lake View, Ill., which he held until 1887, when he was appointed Assistant Engineer for the City of Topeka, Kans., in charge of sewer construction.

In 1888, he entered the service of the City of Chicago, Ill., as Assistant Engineer in the Department of the Bureau of Streets, in charge of street-paving and improvements. From 1895 to 1897, he served as Consulting Engineer on street improvements and sub-divisions. In 1898, he was transferred to the Bureau of Engineering and for the ensuing twelve years he was employed as Assistant Engineer in local charge of the construction of many of the large tunnel and pumping station projects of Chicago.

In 1910, Mr. Samuel temporarily filled the office of City Engineer of Chicago, after which he was appointed Assistant City Engineer. In May, 1911, he became Engineer of Track Elevation, but, in 1914, he returned to the Bureau of Engineering as Assistant Engineer of Water-Works Construction. From 1916 until 1919, he served as Engineer of Water-Works Design and, in January, 1919, was appointed Assistant Engineer in local charge of construction on the Western Avenue and Chicago Avenue Tunnels. He held this position until he retired from the service on July 1, 1930. He then went to Brooklyn, N. Y., where he made his home until his death on November 5, 1930.

Mr. Samuel was a member of the Chicago Athletic Club and of the Municipal Employees Society of Chicago. From an obituary published by the latter is quoted the following tribute to his memory:

"He gave his best to the service of the City of Chicago. Long hours and hard work had no terrors for him, and, up to the day that he left the office for the last time, he maintained close contact with every activity under his supervision."

* * * * * *

"His death ended the career of one of the Bureau's most capable engineers, and there is lost to the City the experience, knowledge, and ability which Mr. Samuel possessed."

Mr. Samuel was elected a Member of the American Society of Civil Engineers on September 6, 1910.

---

[1] Memoir prepared from information on file at the Headquarters of the Society and from data supplied by James J. Versluis, Esq., Chicago, Ill.

IVAN OSCAR SHAFFER, M. Am. Soc. C. E.[1]

DIED JANUARY 25, 1928

Ivan Oscar Shaffer was born in Waterford, Ireland, on July 24, 1887, the son of Isaac and Fanny Wolpert Shaffer. When Mr. Shaffer was seven years old, his family emigrated to America, where he received his early education in the public elementary and high schools. He received the degree of Civil Engineer from the Polytechnic Institute of Brooklyn in 1915.

While Mr. Shaffer was attending school, he was employed in various capacities. From 1906 to 1918 he received his training in various fields of engineering, including drafting, designing, and construction supervision on the Hudson River Tunnels, Pennsylvania coke mines, West Virginia Coke and Coal Company, and on the construction of locks and dams with the War Department; during this period he also was employed in the sewage and water supply branch of engineering with the firm of Hering and Fuller, as well as with George W. Fuller, M. Am. Soc. C. E.; during the latter period up to 1918, Mr. Shaffer was Engineer of Design with the American Can Company, and with Weiskopf and Borroughs in connection with buildings in New York, N. Y.

From 1918 to 1922, he was Chief Engineer of the Fleischmann Construction Company, during which period many of the New York theatres, large motion picture studios, film laboratories, and miscellaneous structures were built under his supervision.

In 1922, he entered the firm of Parker and Shaffer, Engineers and Industrial Architects, in New York City, and during this partnership, the firm was retained to plan the Lloyds Film Storage Building, the Miller Shoe Factory, the Ronzoni Macaroni Factory, the Stanley Patterson Building, the Master Printers Building, and many other structures in New York. In 1926 this firm was dissolved, and Mr. Shaffer practiced as an Industrial Engineer and Architect in the construction of industrial buildings until his death.

Mr. Shaffer was an engineer of varied experience, and a recognized expert in industrial building design. Prior to his death, he was being considered for some of the outstanding building work in New York City. He had established an enviable reputation as an Engineer and Designer, and had the confidence of his clients for his unimpeachable honesty, ability, and dependability.

It is unfortunate that his accidental death cut short a career which had already started in the limelight of success, and would no doubt have been climaxed by a unique and enviable position in his field. Mr. Shaffer is survived by his widow, Bess Shaffer, and a daughter, Muriel Shaffer.

Mr. Shaffer was elected an Associate Member of the American Society of Civil Engineers on October 9, 1917, and a Member on April 16, 1923.

---

[1] Memoir prepared by William Ginsberg, M. Am. Soc. C. E.

STANLEY RUSH SHARTS, M. Am. Soc. C. E.[1]

DIED DECEMBER 5, 1930

Stanley Rush Sharts was born at Dayton, Ohio, on June 18, 1874, the son of Joseph William and Belle (Ealy) Sharts. His father was a lawyer, an associate of the famous Valandingham. Both families were descended from old pioneer stock of that section of Ohio, his maternal grandfather, Dr. Ealy, having been a well-known physician of Dayton's early days.

Mr. Sharts attended the public schools in Dayton and studied civil engineering at the Ohio State University from 1893 to 1897. On the completion of his course at the University, he served, until June, 1898, as Deputy Surveyor of Williamson County, Illinois, where his work consisted mostly of mine locations. At the outbreak of the war with Spain he joined the Second Engineers Regiment of Volunteers and was in Cuba with the United States Army from June, 1898, until May, 1899, engaged in military surveying and mapping. He assisted in laying out the first sanitary sewers in the vicinity of Havana, at Mariano, Cuba.

After his honorable discharge from the Army he returned to Dayton, but very shortly went with the Baltimore and Ohio Southwestern Railroad Company as Instrumentman, in which capacity he remained until January, 1900. In May, 1900, he became connected with the City Engineer's Office in his home city where he was actively engaged until March, 1901. At that time he was appointed Assistant Engineer on ditch and waterway construction in the County Engineer's Office of Montgomery County, Dayton. In September of the same year Mr. Sharts was made Chief Assistant Engineer in charge of the design and construction of bridges and roads. The cost of the steel highway bridges built under his supervision, ranged from $3 000 to $170 000 each. He also designed reinforced concrete slab, girder, and small arch bridges valued at $200 to $8 000 each. From September, 1917, to September, 1919, he was County Engineer in charge of annual improvements the cost of which approximated $600 000. For a short period after leaving the County Engineer's Office, he did general engineering work in and around Lima, Ohio.

In March, 1920, Mr. Sharts left Dayton for Minneapolis, Minn., having received an appointment as Senior Highway Engineer with the Bureau of Public Roads, U. S. Department of Agriculture. For about six months he was assigned to various projects in the States comprising District No. 4, namely, Minnesota, Wisconsin, North Dakota, and South Dakota, but in October, 1920, he was sent to Bismarck, N. Dak., to establish an office of the Bureau. During his nearly eleven years of service there, he saw the highways of North Dakota emerge from a network of trails and a few good roads to many hundreds of miles of a modern Federal highway system.

The large steel bridge spanning the Missouri River between Bismarck and Mandan, N. Dak., was completed at a cost of more than $1 250 000, under his supervision. He also completed the large steel spans at Williston and Sanish, both over the Missouri River, as well as the Roosevelt Memorial

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

Bridge over the Little Missouri at Chaloners, in the Grand Canyon of this river, in the heart of the Bad Lands. All these bridge sites had been ferry crossings before the construction of these modern highway structures under Federal Aid.

In these progressive years when he was in charge of the work of the Bureau of Roads in North Dakota, Mr. Sharts, peculiarly fitted as he was by education, wide experience, personality, and a judicial mind trained to survey all angles before deciding, accomplished far more for the good of the Bureau and the State than many men in the same position would have been able to do in a similar length of time. This work, well known to the public generally in the State and District, will always stand as a monument to the ten years and more of earnest and too strenuous endeavor which he gave to North Dakota.

Mr. Sharts was a member of the Ohio Engineering Society, the Spanish War Veterans, and the American Legion, having held a Captain's Commission in the Reserves. By pressure of official groups in Montgomery County, he was persuaded, with great regret, to resign this Commission, in order to continue his engineering work there. He was also a Thirty-second Degree Mason and Shriner, and had been a Knight of Pythias for many years, as well as a member of the Order of Eagles. He was affiliated with the Delta Tau Delta Fraternity at Ohio State University.

Mr. Sharts was of a quiet disposition, choosing to let his work speak for itself. He was, however, an excellent conversationalist when he felt so inclined. He was a great lover of such sports as hunting and fishing and was a periodical visitor and an indefatigable follower of his favorite pastimes in various parts of Ohio, as well as in North Dakota, South Dakota, and Minnesota. A man who readily made friends and kept them always, he was whole-souled, charitable, and generous to a fault. To his many friends and business associates his loss is irreparable, yet the memory of him and his great work will be an inspiration to those who follow.

He was married in December, 1904, to Bessie Ethel Bone, of Dayton, who survives him.

Mr. Sharts was elected an Associate Member of the American Society of Civil Engineers on December 6, 1915, and a Member on November 26, 1918.

BRONSON HASBROUCK SMITH, M. Am. Soc. C. E.[1]

DIED JUNE 6, 1930

Bronson Hasbrouck Smith, the son of P. Minturn and Ella E. (Phillips) Smith, was born on May 9, 1880, in Brooklyn, N. Y. He attended the public schools in Plainfield, N. J., the Brooklyn Collegiate and Polytechnic Institute, the Cascadilla Preparatory School, at Ithaca, N. Y., and was graduated from Cornell University in the Class of 1901. He was a member of the Zeta Psi Fraternity, and rowed on the Cornell crew.

---

[1] Memoir prepared by Henry R. Kent, Esq., Rutherford, N. J.

After his graduation Mr. Smith secured the position of Assistant to the General Manager of the Perth Amboy Shipbuilding Company, with which Company he remained until it went out of business. He then became associated with the Holland Submarine Torpedo Boat Company, co-operating in the construction, outfitting, testing, and operation of submarines.

From 1904 to 1913 he was identified with Westinghouse, Church, Kerr and Company, starting as Draftsman and advancing to Engineer in Charge. During this period he was associated with many important undertakings, including the Pennsylvania Railroad Terminal in New York, N. Y., and the electrification of the Long Island Railroad.

The next two years were devoted to the railway properties of the United Gas and Electric Engineering Corporation and its allied interests. As Senior Operating Officer of the Jamestown, Chautauqua, and Lake Erie Railroad Company on one of its railroad and steamboat properties at Jamestown, N. Y., he built up the property, improved its operating efficiency, and increased the earnings, so that it was finally sold to competing interests. He then went to the International Railway Company at Buffalo, N. Y., and under Mr. E. G. Connette laid down a new high-speed interurban line between Buffalo and Niagara Falls, N. Y. Later, under Mr. B. F. Wood, then Vice-President, he supervised the design, the placing of contracts, and the construction of steel motor cars for the Memphis, Tenn., and New Orleans, La., properties of the United Gas and Electric Engineering Corporation. He also planned and installed power-house extensions and was concerned with the management of the Richmond Light, Heat, and Power Company, Richmond, Ind.

In March, 1915, Mr. Smith was retained as Senior Engineer in Charge by Henry R. Kent and Company and had responsible charge of the design, purchase, and installation of industrial power plants and manufacturing buildings. He handled three contracts with the Spicer Manufacturing Company, South Plainfield, N. J., including a complete 2 000-kw. power plant, a dam and water supply, and transmission lines. He sponsored three contracts with the American Brass Company, including a 4 000-kv-a. plant at Waterbury, Conn., with transmission line and high-tension sub-station; a power house extension with fire and service water supply at Ansonia, Conn., also a large warehouse at the latter place; a contract with the American Cotton Oil Company for a complete power plant of 2 500-kw. capacity, together with the motorization of its plant at Memphis, Tenn.; besides these were numerous other undertakings, one of peculiar interest being a plant for the recovery of arsenic by the Cottrell Precipitation System.

Mr. Smith left Henry R. Kent and Company in November, 1920, to take a position as Chief Mechanical Engineer with the American Cotton Oil Company, New York, as a result of the Company's satisfaction with the Memphis Plant built by the Kent Company under his supervision. He spent three years modernizing the plants, improving operating conditions, supervising power-plant operation, making extensions and additions to power plants, providing engineering reports, and standardizing purchases of power and mechanical equipment, structures, and supplies.

From August, 1923, to August, 1927, he was Managing Engineer for Meyer and Holler, Engineers and Architects, Los Angeles, Calif. With this Company his duties were largely executive, including engineering, estimating, purchasing, construction, and cost accounting; also work in connection with the negotiation of new contracts and the handling of clients. Outstanding among the many important buildings carried through to successful completion were the Petroleum Securities Building and Grauman's Chinese Theatre in Los Angeles. In the fall of 1927, Mr. Smith severed his connection with Meyer and Holler on account of the recession in building activity in Southern California and devoted his time to consulting work.

His health became impaired in the latter part of 1929, and in November of that year he experienced an attack of Malta fever, followed by a relapse early in 1930. He was never able to regain his strength and passed away on June 6, 1930, in Los Angeles.

Mr. Smith was an engineer of wide experience, an untiring and very productive worker. He possessed to an unusual degree the combination of technical ability and financial sense. He was an enthusiastic yachtsman and was a member of the California Yacht Club and the New York Harbor Yacht Club. He was also a member of the Cornell Club of Los Angeles and the American Society of Mechanical Engineers.

He was married to Florence Merwin, of Brooklyn, on June 19, 1905. Mrs. Smith, with one son, Bronson, and three daughters, Gladys, Evelyn, and Barbara, survives him.

Mr. Smith was elected a Member of the American Society of Civil Engineers on November 26, 1923.

LINTON WADDELL STUBBS, M. Am. Soc. C. E.[1]

Died January 18, 1930

Linton Waddell Stubbs, the son of Frank P. and Margaret (Linton) Stubbs, was born on October 5, 1855, at Thomasville, Ga. His early education was obtained in the local schools at Monroe, La., and at the Brothers' College, at Bay St. Louis, Miss. Later, he entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in 1877.

On the completion of this course, Mr. Stubbs entered the service of the United States Government in its survey of the Missouri River. Later, in 1888, he became Resident Engineer of the Meridian-Shreveport Division of the then Queen and Crescent Railroad System, and continued in that position until 1901. During Mr. Stubbs' service with this System, the road-bed between Delta Point, and Monroe, La., was raised to a grade above the high-water mark in the territory at that time.

In 1902, Mr. Stubbs entered the service of the Southern Pacific Railroad Company. His employment with this Company embraced construction of works of great significance, chief among which was the important bridge across the Atchafalaya River at Morgan City, La.

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

In 1909, Mr. Stubbs resigned from the service of the Southern Pacific Railroad Company to become Engineer in charge of the construction of the Galveston (Tex.) Causeway, a large and important task which was completed in 1912. At this time he retired from the active practice of his profession. In later years, he devoted his time to the supervision of his landed and other interests which developed and grew steadily under his wise and capable direction.

Mr. Stubbs was an Engineer of marked ability and his service was ever characterized by fidelity and devotion to his trust. He was deeply interested in civic affairs and was the author of many forceful and interesting articles relating to the public interest.

He was a Mason, and a member of the Presbyterian Church. Mr. Stubbs leaves a widow, Mrs. Helen (Chase) Stubbs, of Shreveport, La., and two daughters, Mrs. Horace M. Witherspoon and Mrs. J. O. Crawford, both of Los Angeles, Calif.

Mr. Stubbs was elected a Member of the American Society of Civil Engineers on October 3, 1888.

HARRY TAYLOR, M. Am. Soc. C. E.[1]

DIED JANUARY 27, 1930

Harry Taylor, the son of John Franklin and Lydia Taylor, was born at Tilton, N. H., on June 26, 1862. His ancestors were of English stock who came to the United States about 1650 and were among the early settlers of Central New Hampshire. They helped found the Town of Sandbornton Bridge, later called Tilton, and several of them served in the Revolutionary War. Prior to becoming a Cadet at the United States Military Academy, West Point, N. Y., Harry Taylor resided in Tilton and attended the Tilton Academy. He entered the U. S. Military Academy on July 1, 1880, was graduated on June 15, 1884, and was appointed Second Lieutenant in the Corps of Engineers, U. S. Army.

Lieutenant Taylor's first assignment was at Willets Point (now Fort Totten), N. Y., where he served with the Battalion of Engineers and at the Engineer School of Application from 1884 to 1887. In 1887, he was assigned to duty at Wilmington, N. C., where he served as Assistant to the late Brigadier General (then Captain) William H. Bixby, Corps of Engineers, U. S. A., M. Am. Soc. C. E., the District Engineer in charge of river and harbor improvements in North Carolina and South Carolina. He was promoted to First Lieutenant on December 1, 1887. In 1888, he was transferred to duty at the U. S. Military Academy, where he remained about one year, being assigned in 1889 to duty under the orders of Lt.-Col. G. L. Gillespie, Corps of Engineers, U. S. A., then at New York, N. Y., and was placed in local charge of the construction of new fortifications at Sandy Hook, N. J. In 1891, Lieutenant Taylor was transferred to the Pacific Coast where he was placed in local charge of the construction of the lock at The Cascades, Oregon, on the Columbia River. He supervised the erection of the lock-gates, the largest that had been erected to that time, and designed the operating machinery for them as well as many of the masonry details.

---

[1] Memoir prepared by Herbert Deakyne, M. Am. Soc. C. E.

He was promoted to Captain, Corps of Engineers, January 6, 1896, and in the same year was placed in charge of the construction of fortifications on Puget Sound and also of the improvement of waterways in the newly-created Seattle Engineer District, remaining on this duty until 1900. He was then assigned to duty at Boston, Mass., in charge of the construction of fortifications and the works of river and harbor improvement in the Boston Engineer District. He remained there until 1903. Captain Taylor was placed in command of Company L, 3d Battalion of Engineers, in 1903, and took his company to the Philippine Islands in October of that year. He served in the Philippines until October, 1905, and while there was on duty, successively, as Engineer Officer, Department of Luzon; Commander of the 3d Battalion of Engineers; and District Engineer in charge of fortification construction, which was initiated in 1904. He was promoted to the grade of Major, Corps of Engineers, in 1904.

Returning to the United States, Major Taylor, after a few months' duty in the Office of the Chief of Engineers, was assigned, in 1906, to duty at New London, Conn., in charge of river and harbor work in that vicinity and of fortification work at the eastern entrance of Long Island Sound. He was also charged with the purchase and issue of 60-in. searchlights and gasoline-actuated generating sets required for fortifications in the United States and its possessions. The purchasing agency established by him at New London was later moved to Washington, D. C., and during the World War was the procurement agency which purchased a large part of the equipment and materials required by the Engineer Service. It still exists as part of the Supply Section of the Office of the Chief of Engineers.

Major Taylor was promoted to the grade of Lieutenant-Colonel, Corps of Engineers, June 24, 1909. He remained on duty at New London until 1911, when he was transferred to the Office of the Chief of Engineers, Washington, D. C., and was placed in charge of the River and Harbor Section of that office, where he remained until 1916. He reached the grade of Colonel, Corps of Engineers, February 28, 1915. In 1916 he was transferred to duty in New York City, in charge of the work of river and harbor improvement in the First New York District, and as Department Engineer of the Eastern Department, U. S. Army. At the time of Colonel Taylor's assignment to duty in New York, the prospect of the entry of the United States into the World War was growing, and, as a result, in addition to his regular duties, he was engaged in the examination of applicants for commissions in the Reserve Corps and in the selection of sites for camps and for the training of troops.

On May 26, 1917, Colonel Taylor was selected as Chief Engineer, American Expeditionary Forces; he sailed for France on May 28, 1917. He was appointed Brigadier-General, National Army, August 5, 1917, and remained in France until September 12, 1918, initiating the large engineering works required for the support of the American Army in France.

On his return to the United States he was assigned to duty in the Office of the Chief of Engineers, Washington, D. C. He was honorably discharged as Brigadier General, June 1, 1919, and on July 1, 1920, was appointed Brigadier General and Assistant Chief of Engineers. General Taylor on his assignment to duty in the Office of the Chief of Engineers in 1918 was again placed in charge of the River and Harbor Section and continued in this work until his appointment as Chief of Engineers, on June 19, 1924, with the rank of Major General. He served as Chief of Engineers until June 26, 1926, when he was placed on the retired list of the Army, having reached the statutory age of sixty-four years.

General Taylor was awarded the Distinguished Service Medal for his services in the American Expeditionary Forces with the following citation:

> "For exceptionally meritorious and distinguished services. Arriving in France June 11, 1917, as chief engineer, American Expeditionary Forces, he organized and administered the Engineer Department, which included the construction of wharves, depots, railways, barracks, and shelters throughout the theater of operations. He continued these duties with most marked and conspicuous ability, building a complete and efficiently functioning institution."

He was also made a Commander of the Legion of Honor by the French Government for his services during the war.

During his forty-two years' service as an officer of the Corps of Engineers, General Taylor served on many official boards, both in connection with the preparation of plans for the defense of the coasts and in the interest of the improvement and utilization of the rivers and harbors of the United States for navigation and other purposes. He was a member of an Advisory Board of Officers of the Corps of Engineers appointed in 1910 by direction of the President to assist the Secretary of the Interior in connection with granting permission to the City and County of San Francisco to use the Hetch Hetchy Valley in California for a new water supply system.

His experience in fortification construction began with the construction of new works of harbor defense in the early Nineties when Congress, realizing the defenseless condition of the coasts, again began to make appropriations for new fortification construction after a lapse of about twenty years. He was an outstanding fortification engineer, and when a Captain, devised with Capt. R. R. Raymond, Corps of Engineers (now Colonel, U. S. Army, (*Retired*)), an ingenious hoist for transferring the heavy ammunition from the magazine level of the batteries to the firing platform above. This hoist was used to a large extent in batteries constructed prior to the World War. During his long service in the Office of the Chief of Engineers as an Assistant to the Chief, Assistant Chief, and Chief of Engineers, General Taylor's duties were primarily in connection with the improvement and maintenance of rivers and harbors for the purpose of navigation. He acquired an extended acquaintance with the numerous projects adopted or under consideration by Congress, and his intimate knowledge of works of river and harbor improvement, combined with his sound judgment of their value to water-borne commerce, was of great assistance to the committees of Congress in the preparation of appropriation bills for these works. While serving in the Office of the Chief of Engineers he had supervision over many important engineering works for the Federal Government. One of these was the Wilson Dam on the Tennessee River at Muscle Shoals, Ala., in which he was particularly interested and which was completed while he was Chief of Engineers.

General Taylor was an engineer of great ability, with an active, logical mind, and his decisions were made quickly and with excellent judgment. Following his retirement from the Army in 1926, he engaged in consulting practice and was actively occupied with this work at the time of his death.

His youth in his native State had been spent largely in the open, and he had a fondness for hunting and fishing which remained with him all his life. This liking for outdoor life was no doubt largely responsible for his excellent physique, and for his physical appearance which gave little evidence of advancing years. His untimely death from pneumonia, after a brief illness, in Washington, D. C., where he had made his home after retirement, was a shock to his many friends. He was buried in the Arlington National Cemetery where he rests after a life that devoted to his country's service the talents which brought him honor and distinction.

General Taylor was married at Portsmouth, N. H., on October 30, 1901, to Adele Austin Yates, the daughter of Capt. Arthur Reid Yates, U. S. N., of Schenectady, N. Y., and Mrs. Yates, who was Susan Thompson Dwight, of Portsmouth. He is survived by Mrs. Taylor and their two children, Arthur Yates Taylor and Margaret Taylor, now Mrs. Alfred Craven Bruce.

He was a member of the Society of American Military Engineers, and served as its President in 1925. He was also a member of the Army and Navy and Chevy Chase Clubs of Washington.

General Taylor was elected a Member of the American Society of Civil Engineers on October 7, 1896.

LOUIS LINCOLN TRIBUS, M. Am. Soc. C. E.[1]

Died November 9, 1930

Louis Lincoln Tribus was born on May 26, 1865, in Northampton, Mass. He was the son of Louis Tribus, of French descent, and Harriet Jeanette (Kingsley) Tribus, of Westhampton, Mass. His father, at one time a Professor in Amherst College, was an Instructor in a prominent preparatory school, the Willinston Seminary, in Easthampton, Mass. His mother was a descendant from an old Massachusetts family.

After obtaining his early education in private schools on Staten Island, New York, Mr. Tribus entered the Engineering Course of New York University and was graduated in 1885 with the degree of Bachelor of Science and Civil Engineer. In 1888, he received from this University the honorary degree of Master of Science.

---

[1] Memoir prepared by Edward Wegmann, M. Am. Soc. C. E.

After graduating, Mr. Tribus spent two years as Assistant Engineer in the employ of Samuel R. Bullock and Company, of New York (franchise constructors and owners of water-works). The following year he was made Resident Engineer. While in this employ he supervised the construction of water-works for Pensacola, Fla., and Mobile, Ala., and reported on a proposed water-works for Gainesville, Fla., and Tuscaloosa, Ala.

In 1887, he became Assistant Engineer with the late Charles B. Brush, M. Am. Soc. C. E., and was given special charge of water-works design and construction; he was also engaged in many other engineering matters. Mr. Brush had an extensive practice and during the six years Mr. Tribus stayed in his employ he was engaged in making valuations and reports and on the design and construction of water-works and sewers for various places in the States of Ohio, Virginia, Pennsylvania, New Jersey, New York, Kansas, and Missouri. He also made studies for classification yards, traffic handling, and terminal station for the New York and New Jersey Bridge Commission.

From 1887 to 1890, Mr. Tribus was engaged at New York University as Instructor and Acting Professor of Civil Engineering. From 1894 to the end of 1901, he was in private practice as Consulting Engineer in New York, N. Y., reporting on various projects, and designing and superintending the construction of public utilities and water power plants, mainly in the Eastern States, although his practice extended as far as California, Kansas, Nebraska, Ohio, Michigan, and British Columbia.

In 1902, Mr. Tribus went into partnership with Mr. Charles G. Massa, under the firm name of Tribus and Massa, continuing the business of Consulting Engineering which had been so well established by Mr. Tribus.

In this year, he was appointed by the Hon. George Cromwell, who had been elected Borough President of Richmond, as Consulting Engineer and Commissioner of Public Works for this Borough. While in this position, which he held for twelve years, he was at times Acting President of the Borough and Deputy Member of the Board of Estimate and Apportionment, and Alderman. He was a member and Secretary of the New York State Bay Pollution Commission from 1903 to 1906; member of the Board of Consulting Engineers of New York City, and of the Sewer Plan Commission, from 1910 to 1913; and Project Engineer for the United States Housing Corporation, at New London and Groton, Conn., in 1918 and 1919. He was also a member of the following technical societies: The American Society of Municipal Improvements; the Municipal Engineers of the City of New York; the American Water Works Association; and the New England Water Works Association. Mr. Tribus was a good writer and speaker, and had contributed papers to the several societies to which he belonged; he also took part in the discussions of various papers, especially those pertaining to water-works, sewerage, and harbor pollution.

He spent most of his life on Staten Island and took a deep interest in all public and civic matters which affected its welfare. While Commissioner of

Public Works of the Borough of Richmond he was responsible for many engineering improvements that were made, such as the perfection of the highway system on the Island and of trunk and storm relief sewers, as well as the very comprehensive topographical map of the Borough.

He was Past-President of the Staten Island Chamber of Commerce; a Trustee of the Samuel R. Smith Infirmary (now Staten Island Hospital); President of the Tompkinsville Savings and Loan Association; a member of the Staten Island Academy of Arts and Sciences; and Ruling Elder of the First Presbyterian Church of Stapleton. He was a member of the Alumni Association of the New York University and belonged to Psi Upsilon, Phi Beta Kappa, and Iota Alpha (honorary engineering) Fraternities.

Mr. Tribus was an engineer of much ability and of large and varied experience. In the high positions he held for many years, he always acted with dignity, but in dealing with his subordinates he was genial, just, and very kind.

He is survived by his widow, *née* Letitia Hall McCampbell, to whom he was married on October 3, 1899, and by his son, Lucien Hall Tribus.

Mr. Tribus was elected a Junior of the American Society of Civil Engineers on April 4, 1888; an Associate Member on June 1, 1892; and a Member on April 1, 1896.

EDMUND KIMBALL TURNER, M. Am. Soc. C. E.[1]

DIED MAY 6, 1915

Edmund Kimball Turner was born in Marblehead, Mass., on May 11, 1848. His father, Joseph P. Turner, was for many years Cashier of the National Grand Bank of that town, and his mother who died when he was a child, was Mary K. Kimball, of Marblehead.

In September, 1867, Mr. Turner entered the Massachusetts Institute of Technology, then in its infancy, from which he was graduated in Civil Engineering in 1870. He was an enthusiastic student and was one of the Captains of the Institute Battalion, military drill being required at that time, as it is now.

Following his graduation, he was engaged as Assistant Engineer for the City of Charlestown in 1871, and, in 1872, became Assistant Engineer and then Engineer of Construction on the Nashua, Acton, and Boston Railroad, a short line which has since been consolidated with the Boston and Maine System. On the completion of this work he was appointed Chief Engineer of the Fitchburg Railroad, which position he held for seventeen years. In addition, he served during part of this time, as Assistant Superintendent of the Vermont and Massachusetts Division of this railroad.

In 1890, Mr. Turner resigned his position as Chief Engineer with the Fitchburg Railroad Company and entered into private practice as a Consulting Civil Engineer, with an office in Marblehead. He continued in this work until the time of his death. In 1895, he was appointed Consulting Engineer for the Massachusetts Board of Railroad Commissioners on various problems connected with the construction of the Boston Elevated Railway, the appraisal of railroad and street railway properties, and other matters of a like nature, continuing to act as such for practically the remainder of his life. He was active on many important commissions appointed by the Courts, as provided by statute, for the abolition of grade crossings throughout Massachusetts, and was engaged by the Fitchburg Railroad Company in a consulting capacity, making annual inspections of the property. He also advised its officers with reference to maintenance and needed improvements.

---

[1] Memoir prepared from information on file at the Headquarters of the Society and from the memoir prepared by the late George F. Swain, Past-President and Hon. M. Am. Soc. C. E., *Journal*, Boston Soc. of Civ. Engrs., November, 1916, p. 477.

Mr. Turner was a member of the Institute of Consulting Engineers and of the Boston Society of Civil Engineers which he joined in 1874.

He was a man of wide experience, of well-balanced judgment, and of unlimited information on transportation matters. His knowledge of European as well as American practice may be accounted for by his annual trips abroad for study, observation, and recreation during a period of nearly twenty-five years.

Mr. Turner was of a reserved nature, undemonstrative, and slightly diffident. He was a strict disciplinarian, with himself, as well as with others, and an individual of decided and definite ideas. He was well read in a wide variety of subjects and a great lover of art and music and of all that was beautiful in Nature.

He was held in high esteem by members of the Legal Profession with whom he frequently had occasion to work. Many of his personal qualities made him an excellent witness and during his professional career he often appeared before Courts or other legal tribunals. Regarding his less known services, those in other than the engineering field, one of his lawyer friends writes as follows:

> "Entirely apart from Mr. Turner's services in the strict lines of his profession, he was much sought after by his friends and others for advice on legal and business matters and on affairs of property interests. His selection as executor of the William C. Todd estate, involving a very considerable sum of money given for educational purposes and as to which there were several legal controversies, is an illustration of how he was regarded by one of the early business men of New England. Those who had occasion to consult him always obtained a sympathetic hearing and definite advice that was always honest and sound, though perhaps not always in accordance with their desires."

Mr. Turner was elected a Member of the American Society of Civil Engineers on November 4, 1891. He also served as a Director of the Society from 1899 to 1901.

CHARLES AUGUSTUS VAN KEUREN, M. Am. Soc. C. E.[1]

Died April 21, 1930

Charles Augustus Van Keuren was born in New York, N. Y., on August 23, 1852. In his early years he moved to Jersey City, N. J., where he attended the public schools and received his primary education. He then attended Hasbrouck's Institute, in Jersey City, and afterward took a Special Course in Civil Engineering.

---

[1] Memoir prepared from information supplied by N. D. Wortendyke, Assoc. M. Am. Soc. C. E., and on file at the Headquarters of the Society.

In 1873 and during part of 1874, Mr. Van Keuren was employed in the office of John P. Culver, Civil Engineer and Surveyor, of Jersey City, as an Assistant in general surveying and map work. In 1874 and 1875, he served as Rodman in the offices of the New Jersey and New York Railroad Company, and from 1875 to 1880, he was with the Pennsylvania Railroad Company in its Transportation Department. On March 30, 1880, he was appointed Clerk to the Chief Engineer, in the Engineering Bureau of the Board of Public Works of Jersey City; he resigned from this office on June 6, 1881.

From 1881 to 1884, Mr. Van Keuren served as Transitman and Assistant Engineer with the West Shore Railroad Company (Buffalo Division) and assisted in the preliminary survey and location of the line from Buffalo to the Genesee River (about 65 miles). He made the final location, and was in charge of a 9-mile section from Akron, Ohio, west; later, he was placed in charge of an additional 9-mile section. On the completion of the railroad, he was appointed Engineer of Maintenance of Way, with headquarters at Buffalo, N. Y. Subsequently, he was employed as Assistant Engineer with the Erie Railroad Company at Jersey City.

In 1884, Mr. Van Keuren was appointed Assistant Engineer with the Municipality of Jersey City, which position he retained until January, 1898, when he was made Chief Engineer of the same municipality, serving as such until his death. His duties consisted of the superintendence of all classes of municipal work and various improvements.

Mr. Van Keuren was a member of the Masonic Order, a Knights Templar, and a member of the Shrine, the Jersey City Lodge of the Benevolent and Protective Order of Elks, the Holland Society, of New York, the National Water Works Association, and the New Jersey State Board of Professional Engineers and Land Surveyors. He was also Vice-President and Trustee of the New York Bay Cemetery in Jersey City. He is survived by his widow and a son.

Mr. Van Keuren was elected a Member of the American Society of Civil Engineers on May 3, 1905.

ROBERT LAWRENCE VAN SANT, M. Am. Soc. C. E.[1]

Died April 4, 1930

Robert Lawrence Van Sant was born in Jefferson County, Virginia, on February 11, 1853.

Mr. Van Sant's engineering service was begun in 1870 as Rodman in the Engineer Corps on the Hannibal and Central Missouri Railroad. In 1871, he was engaged as Assistant Engineer on surveys and construction for the Sny Island Levee in Illinois. From 1873 to 1875, he was employed on railroad surveys and also conducted a general practice in Missouri, Kansas, and

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

Colorado. The following year he acted as Assistant Engineer on the Union Depot Extension of the St. Louis, Kansas City, and Northern Railway from St. Louis to Ferguson, Mo.

In 1876, Mr. Van Sant again entered private practice in Missouri, Kansas, and Illinois, and at the expiration of a two-year period he was appointed Chief Engineer for the Memphis, Kansas, and Colorado Railroad Company, on location and construction. In 1879, he was employed as Assistant Engineer for the St. Louis and San Francisco Railroad Company. In this capacity he was in charge of location and construction in Kansas, Missouri, and Arkansas, as well as of a survey through Indian Territory, between Fort Smith and Paris, Ark. In 1884, he made surveys in Missouri and Illinois and was engaged on a contract for repairing the Memphis and Little Rock Railroad between Forrest City and Memphis, Tenn., which had been severely washed away in the floods of 1882, 1883, and 1884.

During this period he was located in Memphis where he was also engaged in general engineering and contract work. His several engagements during successive years were as follows: Construction of about 30 miles of the St. Louis, Troy, and Easton Railway, a coal road in Illinois; 1886, contracting and bridge building on an extension of the St. Louis, Peoria, and Northern Railroad, in Illinois, in partnership with Mr. R. P. Lutes; the rebuilding of the Howe truss span for the Atchison, Topeka, and Santa Fé Railroad Company, in Kansas; 1887, Chief Engineer, the St. Louis, Arkansas, and Texas Railroad (Cotton Belt), changing the gauge from 3 ft. to 4 ft. 8½ in.; 1892 to 1904, Contractor engaged on river-protection work in the Middle West for the Wabash and Santa Fé Railroad Companies; and 1904 to 1909, engaged in private practice as a Contractor, with an office in St. Louis. This period marks Mr. Van Sant's actual retirement from active business.

He was a bachelor, and after his retirement lived a quiet and secluded life in St. Louis, until his death on April 4, 1930.

Mr. Van Sant was elected a Member of the American Society of Civil Engineers on September 3, 1884.

HARRY LEE VAN ZILE, M. Am. Soc. C. E.[1]

DIED JANUARY 6, 1931

Harry Lee Van Zile was born at Troy, N. Y., on September 20, 1864, the second son of Oscar Edward and Sarah (Perry) Van Zile. As a boy, he exhibited marked ability in mathematics and entered the Rensselaer Polytechnic Institute at a comparatively early age. The combination of insight and industry that had enabled him to stand well in his elementary schools made his career at the Institute notably successful and obtained for him, after he had received the degree of Civil Engineer, the position of Assistant to the Director of the Institute and Instructor in Surveying and Map-Drawing.

In 1885, his father, Oscar E. Van Zile, had founded a Company, with a plant at Albany, N. Y., to manufacture water-tube boilers and small iron and steel bridges. This concern eventually became the Franklin Boiler Works Company. On the death of his father, Harry Lee Van Zile took up his residence in New York, N. Y., and established the Van Zile Ventilating Corporation, the output of which consisted of an air-deflecting apparatus which he had invented and patented.

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

Mr. Van Zile, in 1914, had reached middle age without suspecting that there was to come to him a clarion call for participation in the first world-wide war in which mankind had as yet indulged. When the United States entered the conflict in 1917, it became immediately apparent that the Government was in great need of constructing engineers technically equipped to overcome the stupendous obstacles confronting them, and physically fitted to endure the hardships that service of this nature engendered. That President Wilson was fully justified, at this crisis, in conferring upon Mr. Van Zile the rank of Major of Engineers subsequent events proved. The more spectacular features of a great war are bound to overshadow the achievements of those whose devotion and technical skill make it possible for an army to function at the front as a fighting force. However, future historians, it may safely be predicted, will be inclined to give high praise to the Engineering Department of the American Expeditionary Force for what it accomplished, against almost insuperable difficulties, in rendering 2 000 000 comparatively untrained American combatants eventually effective as decisive factors in the final battles of the first World War.

In this memoir only a general outline can be given of the activities in which Major Van Zile, with headquarters at Tours, France, was engaged during those crucial months in 1917 and 1918, when Yankee ingenuity performed the miracle of transferring an army of fighting men across 3 000 miles of water, beneath which lurked enemy submarines, with the loss of only a few combatants. "The Historical Report of the Chief Engineer", published by the United States Government in 1919, makes repeated references to Major Van Zile's distinguished services as a constructing engineer at Tours. His task is described as "technical supervision of the construction of oil-storage facilities and of various plants." Refrigeration plants and electrical and mechanical installations came under his supervision. Regarding one feature of the gigantic operations devolving upon Major Van Zile at that crucial juncture, the official report states:

> "In the formation of the division of construction and forestry, which succeeded to the work of the chief engineer, line of communications, all engineering, both design, supervision and construction in connection with electric power, was allotted to an electrical-mechanical section which had been consolidated with other sections into the plant construction division under Major Van Zile."

These quotations from the Chief Engineer's official report, a publication that will be of inestimable value to future historians, give merely a hint as to the varied, exacting, and supremely important activities in which Major Van Zile was engaged during the two years of his war service in France. That his energy, ability, enthusiasm, and technical skill were of very real service to the cause of the Allies was a fact recognized officially by both the Government of the United States and that of France. In 1919, the latter country conferred upon him the "Palmes d'Officier de l'Instruction Publique et des Beaux Arts."

Major Van Zile was a member of the Engineers Club of New York. Although he was a bachelor, he was not especially devoted to club life. He had the faculty of making and keeping friends, but was not what is somewhat loosely called "a good mixer." He was absorbed in the technical problems that confronted him both as an inventor and manufacturer, but was inclined, in his hours of leisure, to broaden his mind by omnivorous general reading, displaying always a special fondness for books dealing with historical and biographical subjects. He was also greatly interested in the recent scientific generalizations by which Einstein, Jeans, Eddinger, Millikan, and others have awakened the interest of the public at large in problems and mysteries that have come within the purview of contemporary commentators on recent scientific progress.

Such in its simple outlines is the story of a life that was justified by unselfish and worthy achievement. In this connection it would be unfair to the memory of Harry Lee Van Zile not to stress the kindly, sympathetic, and self-sacrificing qualities that endeared him to all who had the privilege of knowing him intimately. Following his death on January 8, 1931, one who knew him well paid to him a tribute in verse, the last stanza of which may well be quoted here in connection with the foregoing:

"His soul was as the souls of humans are
Whose minds obey the promptings of the heart.
He passed anon to seek a beckoning star—
Who loves his kind pursues God's greatest art!"

Mr. Van Zile was elected a Junior of the American Society of Civil Engineers on January 6, 1886, and a Member on October 8, 1918.

EDWARD FRANKLIN VINCENT, M. Am. Soc. C. E.[1]

DIED DECEMBER 29, 1923

Edward Franklin Vincent, the son of Charles Floyd and Hannah (Reynor) Vincent, was born at the farm home near Maquoketa, Jackson County, Iowa, on July 11, 1863. In 1869, his parents moved to Mason City, Iowa, where his father, a Civil Engineer, held the office of County Surveyor for Cerro Gordo County from 1870 to 1888. Having acquired an early education in the public schools and having been graduated from the Mason City High School in 1880, Mr. Vincent entered the Iowa State University, from which he received the degree of Civil Engineer in 1884.

After teaching two years in the Mason City schools, he began his professional career in the spring of 1886 when he was employed by the Union Pacific Railroad Company in Wyoming. In the fall of 1887 Mr. Vincent accepted a position as Engineer for the contractor engaged in constructing the State Capitol Building at Denver, Colo., and continued in that capacity until 1889, when he again began a short period of service with the Union Pacific Railroad Company on surveys in Utah, Wyoming, California, and Nevada. At the close of this period he was placed in charge of a field party for the United States Geological Survey on the head-waters of the Arkansas River, near Leadville, Colo.

---

[1] Memoir prepared by a Committee of the Colorado Section, consisting of R. C. Gowdy, Arthur O. Ridgway, and the late Herbert L. Aulls, Members, Am. Soc. C. E.

During the decade from 1889 to 1900, Mr. Vincent served the City of Denver in the capacity of Engineer on public improvement work, and for six years of this period he was Principal Assistant Engineer for the Board of Public Works. From 1900 to 1904, he occupied the position of Resident Engineer for the Fort Worth and Denver City Railway Company, and during this time he made his home in Fort Worth, Tex. In 1904, he was Assistant Chief Engineer for the Wyoming State Railway Company which was subsequently absorbed by the Chicago and Northwestern Railway Company. Later in 1904 he was Assistant Engineer on location for the Missouri Pacific Railway, and afterward had charge of the drafting forces of that Company in St. Louis, Mo. He returned to Colorado in 1906 as Assistant Chief Engineer of the Colorado and Southern Railway Company with headquarters in Denver, where he also made his home. In 1915, he succeeded the late Herbert W. Cowan, M. Am. Soc. C. E., as Chief Engineer of this Company, in which position he remained until March 1, 1919, when he was assigned to special engineering duties with the title of Special Engineer.

Eminently well fitted by years of experience with its pertinent characteristics, as Chief Engineer and Special Engineer of the Colorado and Southern Railroad Company, Mr. Vincent gave unstintingly of his best efforts to its welfare. After years of hard work in the discharge of his exacting duties and with few if any vacations, he was persuaded during the holidays at the close of 1923 to avail himself of a few weeks' rest and recreation in Florida. It was while on his journey there that illness overtook him, his death occurring December 29, 1923, at Jacksonville, Fla. His remains were returned to and interred at his home town, Mason City.

Mr. Vincent's outstanding characteristics were conscientious devotion to his chosen profession, and his thorough and painstaking care, even to the minutest detail, in all his engineering accomplishments. As a Mason he was a member of Oriental Lodge No. 87, A. F. and A. M. He received the Royal Master Degree, Denver Council No. 1, R. S. M., February 28, 1916, was greeted as a Select Master on the same date, and was chosen as a Super-Excellent Master, May 22, 1916. He was one of the earliest members of the Colorado Society of Engineers, and was also a member of the Colorado Engineering Council during the period 1916-1921, representing thereon the Colorado Section of the Society, of which Section he served as President for one term.

He was married June 1, 1892, at Mason City to Myrtie Ann Gregory, who, with a son, Gregory A., survives him.

Mr. Vincent was elected a Member of the American Society of Civil Engineers on August 31, 1909.

ALBERT LOWRY WEBSTER, M. Am. Soc. C. E.[1]

DIED MARCH 24, 1930

Albert Lowry Webster was born in Orange, N. J., on August 3, 1859. His father was Benjamin Crampton Webster, President of the Lehigh Zinc Company, in Bethlehem, Pa., and also of John Jewett and Sons until this firm was merged with the National Lead Company of New York, N. Y. He had acted as Private Secretary to Governor Morgan of New York State during a part of the Civil War. His mother was Eliza Campbell (Wilbur) Webster, the daughter of Marcus and Catharine (Campbell) Wilbur, of Orange.

During his early youth Mr. Webster attended the Moravian School in Bethlehem. He received his preparatory education at Holbrook's Military Academy (the Briarcliffe School), at Ossining, N. Y., from which he entered the Sheffield Scientific School of Yale University. He was graduated from Yale in 1879, with the degree of Bachelor of Philosophy, receiving honorable mention for excellence in physics and the unusual merit of his thesis. In 1892, he received the degree of Civil Engineer. Mr. Webster subsequently enrolled as a graduate student at Yale, at the same time acting as Assistant Instructor in Surveying and Drawing. He was a member of Alpha Chi and Berzelius Fraternities.

From 1881 to 1884, he was connected with the United States Geological Survey as an Assistant Topographer in Utah, New Mexico, and Nevada; from 1884 to 1885 he was a Fellow by courtesy at the Johns Hopkins University, at Baltimore, Md., and from 1885 to 1886, he was engaged chiefly in the office of W. H. Lindley, Consulting Engineer, and Stadtbaurat, at Frankfurt-am-Main, Germany, acting as an Associate in the design of the sewage disposal plant and water-works of that city. During this time, Mr. Webster also designed the famous water tower in Warsaw, Poland.

In 1887 he returned to New York, where he opened an office for the private practice of his profession, engaging in consulting civil and sanitary engineering work which he continued until his retirement in 1926. During this period he also served in the following capacities: From 1888 to 1889, Expert Sanitary Engineer for the Department of Public Works of the City of New York; 1897, on the Board of Organizers of the Tenement House Department; 1901, expert for the Tenement House Commission, and, subsequently, Consulting Engineer for the Property Owners' Committee in the matter of the slide in the Park Avenue Subway Tunnel, for the construction of the water main and sewers in the extension of Riverside Drive, and for many New York buildings, including the Grand Central Terminal; the New York County Courthouse; the Postal Telegraph Building; the Bellevue Hospital group; the Public Library; the Woolworth, Municipal, and Equitable Buildings; Barnard College; the Brooklyn Young Men's Christian Association; Mount Sinai Hospital; and the New York Infirmary for Women and Children. Later, he served as Consulting Expert in connection with various projects elsewhere in the country, such as the construction of the sewerage plant at Hampton Institute, Hampton, Va., the Esmeralda Water Company, at Goldfield, Nev., the Carnegie Building, and the Carnegie Institute, at Pittsburgh, Pa.

---

[1] Memoir prepared with the assistance of Prof. Duane Reed Stuart, Princeton, N. J., and from data on file at the Headquarters of the Society.

Incidentally, from 1893 to 1894, Mr. Webster was a Special Student in Architecture at Columbia University, and, from 1900 to 1901, Lecturer in Sanitary Science at the Cornell Medical School in New York. About this time, he also directed the design and construction of the sea-wall for the Westchester County Land Association, as well as systems of water supply, sewage disposal, sewerage, drainage, under-drainage, and plumbing for institutions, private properties, and similar structures.

Mr. Webster was a member of the American Public Health Association, the American Association for the Advancement of Science, the Philosophical Society of Washington, the Franklin Institute, the Century Association, and the Yale Club of New York. During the World War, he served as Chairman of the Committee on Drinking Water for the Sanitary Committee of the Council of National Defense. From 1917 to 1920, and from 1922 until 1924, he was a member of the Executive Committee of the Yale Engineering Association. He was also a member of a Sub-Committee on Plumbing of the Building Code Committee for the United States, which was appointed in 1921 by President Hoover, then Secretary of Commerce.

He was the author of various reports and monographs for the United States Geological Survey, as well as "Principles of Modern Plumbing Practice" (1901), and "Caleb and the Friendly Animals", a collection of children's stories, with illustrations by himself, published in 1928. He also contributed articles on technical sanitary subjects to various scientific periodicals.

Mr. Webster's death occurred on March 24, 1930, from angina pectoris, and he was buried in Rosedale Cemetery, at Orange, N. J.

On June 1, 1897, he was married in New York City to Mary Say, a daughter of Cyrus Jay Lawrence and Emily Amelia (Hoe) Lawrence. Mrs. Webster, together with two children, Benjamin Lawrence and Aileen Webster, survives him.

Mr. Webster was elected a Junior of the American Society of Civil Engineers on September 6, 1882; an Associate Member on June 3, 1891; and a Member on April 6, 1909.

JAMES EDWARD WHITFIELD, M. Am. Soc. C. E.[1]

Died November 4, 1930

James Edward Whitfield was born in Albany, N. Y., on September 27, 1859, the son of Robert P. Whitfield, a well-known geologist and paleontologist, who was for many years Curator of the Department of Geology of the American Museum of Natural History, in New York, N. Y. Mr. Whitfield's education was received at the Rensselaer Polytechnic Institute, Troy, N. Y.

---

[1] Memoir prepared by Frederick Wynkoop, Esq., Philadelphia, Pa.

From 1880 to 1888 he was connected with the United States Geological Survey, under A. A. Blair, at Newport, R. I., and with Rafael Pompelli in the Northern Transcontinental Survey for the Northern Pacific Railroad, after which he went to Washington, D. C., deserting the field of engineering for that of chemistry. He analyzed many of the new minerals found in the West and numerous analyses by him were published in the Survey reports and copied by James Dwight Dana in his "System of Mineralogy". Mr. Whitfield was considered an authority on the analysis of meteorites, and made many of the analyses for Dr. George P. Merrill, of the Smithsonian Institution, Washington, in his great work on meteorites. While in Washington Mr. Whitfield became Professor of Chemistry in the National College of Pharmacy from which institution he was granted the degree of Doctor of Philosophy.

In 1888 he became a partner in the firm of Booth, Garrett and Blair, Analytical and Consulting Chemists, Philadelphia, Pa., of which firm he was Senior Partner at the time of his death. While with this firm he gained considerable reputation in metallurgical analysis and the investigation of mining and cement properties throughout the United States.

On December 25, 1885, Mr. Whitfield was married to Florence P. Morton, who died in 1928. They had no children. He left the income from his estate to a favorite sister-in-law, upon whose death the principal goes to the American Philosophical Society, Philadelphia, in which he took great interest. He was also a member of the American Chemical Society, the Franklin Institute, and the American Association for the Advancement of Science.

He had not been well since the death of Mrs. Whitfield; his death, however, was a great shock to all his friends and associates. He was a great fisherman, spending a number of weeks every winter in tarpon fishing in Florida. His unfailing good nature and geniality to all with whom he came in contact will cause him to be missed greatly by those who were privileged to know him.

Mr. Whitfield was elected a Member of the American Society of Civil Engineers on March 2, 1904.

FRANK CLINTON WIGHT, M. Am. Soc. C. E.[1]

DIED SEPTEMBER 18, 1927

Engineer and journalist of rare ability, Frank C. Wight, Editor-in-Chief of *Engineering News-Record,* died on September 18, 1927, at the age of 45 years, when he had only just entered upon a career of wide and influential service to the profession. His qualities of mind and spirit, his prodigious capacity for work, as well as his character as a man, all promised that he should become one of the notable figures of the Civil Engineering Profession in his generation.

---

[1] Memoir prepared by F. E. Schmitt and Willard T. Chevalier, Members, Am. Soc. C. E.

The story of his career is soon told, for nearly the whole period of his manhood was devoted to the paper he served and revered. Nor can that story portray his individuality and powers. These live in the memories of his friends as his true memorial.

Born in Washington, D. C., on February 26, 1882, Frank Clinton Wight received his technical education at Cornell University (1904), and then entered the Bridge Department of the District of Columbia, under Walter J. Douglas, M. Am. Soc. C. E. A large bridge program was then in execution, and work on both steel and concrete structures, notably the Piney Branch, Anacostia, and Connecticut Avenue Bridges, gave Mr. Wight a firm grounding in the practical side of engineering methods. Even at that time his work revealed clearly that he would rise to prominence in whatever field of engineering he might enter.

After less than three years came an opportunity to join the Editorial Staff of *Engineering News,* then under the able direction of the late Charles Whiting Baker. In the new activity Mr. Wight's work won instant approbation from his superiors and colleagues, not only because of the technical and journalistic abilities that it revealed, but even more because of his skill in planning and administration. It was this latter quality that led to his being appointed Managing Editor in 1913, when the growth of the paper compelled Mr. Baker to relinquish part of his duties. Mr. Wight met the increased responsibilities even more capably than had been expected, and because of his great working capacity he managed, not merely to direct the business of the paper with high efficiency, but also to continue his technical work undiminished.

In 1917 the consolidation of *Engineering News* with *The Engineering Record* to form *Engineering News-Record* brought E. J. Mehren, M. Am. Soc. C. E., to the editorship of the new paper. Four years later, however, the new Editor enlisted Mr. Wight's administrative skill by placing him in operating charge as Managing Editor. In 1923, when Mr. Mehren was advanced to an executive position, Mr. Wight was the unanimous choice of officers and staff to succeed him as Editor-in-Chief. In this position of leadership he raised the paper to increased strength and influence. His broad technical knowledge and keen insight, his journalistic intuition, and his distinctive personality, were equally effective in the daily tasks of the editorial sanctum, staff and company councils, and outside contacts.

Less than four years were granted him to serve as leader of his paper. Yet in that short time he contributed fundamentally to enhance its strength and to lay a solid foundation for its future progress. In addition, he was active outside the paper, in both technical and journalistic fields. He served on numerous committees of the American Society of Civil Engineers, the American Concrete Institute—the *Proceedings* of which he edited for some years—the New York Editorial Conference, and the National Conference of Business Paper Editors. The two last-named organizations he served as President. He was a member also of Secretary Hoover's Committee on Highway Traffic Safety, and Chairman of the National Fire Protection Association's Committee on Fire Protection of Construction Operations.

Rugged and active, Mr. Wight was none the less of so intense a nervous organization that overwork finally took its toll and in 1927 a threatened nervous collapse compelled him to take several months' rest. With characteristic impatience to be about and doing, he insisted on returning to the Editor's chair before his vitality had been fully re-established. Soon after his return a throat infection took hold of him and within less than a week he had succumbed.

Those who had been longest associated with him voiced appreciation of their friend and leader in a record of personal impressions published within a few days after his death in the paper he had just left. A few brief quotations from this tribute[2] will tell most eloquently of his rank among his fellows:

"An active brain, a penetrating vision and fearless courage are the characteristics of the great editor, and with them must come that undefinable journalistic intuition that goes beyond. Wight possessed all these. In lore and in keenness, in breadth of sympathies and in understanding, he was one of the elect few. In vision he was at once the dreamer of possibilities and the prophet of coming realities. In courage he was supreme.

"Journalism has lost as much as has our paper itself. It is an art that flourishes on the ideals contributed by strong leaders. Wight brought to it unfaltering ideals and a forceful personality; through them he was an inspiration to others. And even farther abroad he was an influence for progress. The larger journalism of public information and opinion was gaining strength from him. That influence was destined to become much stronger, and to help bring back to honor in these hurrying days some of the old-time editorial virtues.

"An engineer of brilliant talents, a champion of all that was constructive, an enemy of debasing tendencies, he was sorely needed by the profession. His influence had to be exercised mainly through the pen. But perhaps never was such influence more valuable than at present, when engineering as a profession is crossing paths with industrial growth and evolution. Wight discerned clearly the new complications then arising, and foresaw the many changes in engineering life which it must bring. He appreciated as well how human conditions and ways are changing as life comes to depend ever more intimately on the engineer's planning. He shared in every movement and effort on which the future of the profession must be erected."

The spirit and substance of the foregoing lines best portray Mr. Wight's great service to civil engineering. He was a virile leader in the work of recreating the Engineering Profession to serve most effectively the new mechanism of industrial and business co-operation. Although by temperament an individualist, he perceived the need for efficiently planned joint effort so clearly that all his endeavors were enlisted in its behalf. Struck down prematurely, his striving to this end had only begun; yet by virtue of his ability to inspire others with his ideals and convictions, his influence will persist for years to come as a potent factor in behalf of sound professional progress.

Mr. Wight was married in 1911 to Julia T. Welles, of New York, N. Y., who, with four children, surives him.

Mr. Wight was elected a Member of the American Society of Civil Engineers on April 3, 1922.

---

[2] *Engineering News-Record*, September 22, 1927.

FRANK MARTIN WILLIAMS, M. Am. Soc. C. E.[1]

Died February 20, 1930

Frank Martin Williams was born on April 11, 1873, in Durhamville, N. Y., the son of William and Ellen L. (Sterling) Williams. He was of Welsh and Mohawk Dutch descent. On his mother's side he was of the fifth generation removed from Henrich Starin, the first Judge of Herkimer County, New York, and an officer in the American Army at the Battle of Oriskany in the Revolutionary War.

Mr. Williams' schooling began in the district school at Durhamville. Afterward, he attended the Oneida, N. Y., High School, from which he was graduated in 1890. In the fall of 1891, he entered Colgate University where he became a member of the Phi Gamma Delta Fraternity. He was graduated from Colgate in 1895 with honors and with membership in Phi Beta Kappa.

After graduation, Mr. Williams accepted employment with the City of Oneida as an Engineer on Highway and Sewer Construction, and spent his spare time in the study of law in the office of the late J. T. Durham in that city. Believing that a full course in law would be of assistance to him in the Engineering Profession, he entered the College of Law at Syracuse University and was graduated with the degree of Bachelor of Laws in 1897, and admitted to the Bar.

In December, 1897, he entered the service of the New York State Engineer and Surveyor as Rodman and remained until April, 1898. In November, 1898, Mr. Williams became Resident Engineer for the Stanwix Engineering Company, of Rome, N. Y., in charge of the construction of water-works and electric light plant at Charlotte, N. Y.

In April, 1900, he again entered the engineering service of the State of New York, advancing, successively, from the grade of Rodman, through that of Leveler and Assistant Engineer to Resident Engineer in charge of maintenance and construction of highways in various counties of the State.

In November, 1908, Mr. Williams was elected State Engineer and Surveyor of New York State, serving a two-year term from January, 1909, to December, 1910, inclusive. During this term as State Engineer and Surveyor, certain canals of the State were in process of improvement, the project being commonly known as "The Barge Canal Improvement." Under his supervision and direction, plans and estimates were prepared and contracts were let for more than $30 000 000 worth of work; and $14 000 000 worth of construction was completed. During 1910 and 1911, Mr. Williams was also Chairman of the Barge Canal Terminal Commission and, in that capacity, in the summer of 1910, inspected the harbors of Europe.

In June, 1911, he became Chief Engineer of the Coleman-du Pont Road, Inc., in charge of surveys, plans, and estimates for the proposed du Pont Boulevard in the State of Delaware. In 1912, Mr. Williams was appointed Chief Engineer of the Portage County Improvement Association, with headquarters at Ravenna, Ohio. This work involved the supervision of an extensive mileage of highway improvement in Eastern Ohio.

---

[1] Memoir prepared by Roy G. Finch, M. Am. Soc. C. E.

In November, 1914, Mr. Williams was again elected to the office of State Engineer and Surveyor in New York and, with three subsequent re-elections, served from January 1, 1915, to December 31, 1922, inclusive. During these eight years, many of the most difficult problems connected with the Barge Canal construction were solved, including the settlement of the railroad-crossing question, and the location, design, and construction of terminals. The entire Barge Canal System was opened for service on May 15, 1918.

From January, 1923, to the date of his death, Mr. Williams was associated with Roy G. Finch and Edward H. Sargent, Members, Am. Soc. C. E., as Consulting Engineer, with offices in Albany, N. Y.

He was a Consultant for the New York State Bridge and Tunnel Commission and the New Jersey Interstate Bridge and Tunnel Commission on the construction of the Holland Vehicular Tunnel under the Hudson River, connecting New York, N. Y. with Jersey City, N. J. He was also Consultant for the Hudson River Regulating District on the construction of the Sacandaga Reservoir; for the Water Service Commission of the City of Oswego, N. Y., on the construction of a hydro-electric development at Dam No. 6, Oswego River; and for various public utility corporations.

Mr. Williams was a member of the American Institute of Consulting Engineers, New York State Society of Professional Engineers, and an Honorary Member of the Albany Society of Engineers.

Colgate University, his Alma Mater, honored him in 1915 with the degree of Doctor of Science, and he was a Trustee of that institution for more than twelve years prior to his death; and, in addition, for three years, he served as President of Colgate University Alumni Association. In 1919, Syracuse University conferred upon him the degree of Master of Civil Engineering. His latest honor came only a short time before his death, when President Hoover appointed him a member of the Interoceanic Canal Board.

In politics, Mr. Williams was a staunch Republican, and his life was marked by clean achievement in that field. His personality and his conduct in office inspired confidence.

He was a member of the First Presbyterian Church, Albany, of the Masonic Order, in Goshen, N. Y., and of the Benevolent and Protective Order of Elks, in Oneida.

In 1907, he was married to Lucy Mary Sterling, of Watkins Glen, N. Y. They had two sons, Frank Martin Williams, Jr., who died in infancy, and David Sterling Williams.

The life of Frank M. Williams comprised a variety of interests, but whatever he attempted he did thoroughly. He was probably best known as the New York State Engineer and Surveyor who was elected to that office five times and during whose administration the greater part of the Barge Canal Improvement was planned, constructed, and completed.

His life has been well outlined and epitomized by one whose acquaintance and friendship covered college days and extended to the time of his death, Dr. A. W. Risley, Professor of History in the New York State College for Teachers, Albany, who is quoted,[2] as follows:

> "As we think of Frank Williams, devoted son of Colgate, we are sure that three qualities jut out boldly. These are common sense, silent fighting purpose, and reliability. Of these, the last was most evident in his entire career. Diligence, more than brilliance. Reliability, not display."

Mr. Williams was elected a Member of the American Society of Civil Engineers on May 28, 1912. He served as Director of the Society from District No. 3 for the term, 1926-1928, and was also a representative from the Society on American Engineering Council.

WILLIAM FISH WILLIAMS, M. Am. Soc. C. E.[1]

Died October 1, 1929

William Fish Williams, the son of Captain Thomas W. and Eliza Azelia (Griswold) Williams, was born on the whaling ship, *Florida,* in the South Pacific Ocean, somewhere between Tasmania and New Zealand, on January 12, 1859, in a howling gale of wind. The first seven years of his life were passed aboard the various vessels of which his father was Master, accompanied by his mother and a younger sister, who was also born at sea.

He received his early education in the schools of Oakland and San Francisco, Calif. He was graduated from Columbia University, New York, N. Y., in 1881 with the degree of Civil Engineer, and, in 1882, with that of Mining Engineer.

During 1883 Mr. Williams was engaged in examining mines in Colorado, Utah, and California, and the years from 1884 to 1888 were spent in private practice as a Civil Engineer in Connecticut. From 1890 to 1893 he served as Chief Engineer of Cumberland Lands, Limited, in Stewart County, Tennessee, a property of 46 000 acres of farming, timber, and mineral lands. His work during this period included a survey of the entire property, laying out of town sites, and the location of 15 miles of railroad, as well as designs for the mechanical work of the plant.

From 1893 to 1912, he filled the office of City Engineer of New Bedford, Mass., during which time he represented the City as Engineer in supervising the elimination of railroad grade crossings in that municipality. Mr. Williams likewise served as Chief Engineer of the Fairhaven Bridge constructed under his direction at a cost of $1 250 000. The intercepting sewer system was also under his direction for two years after he went to the State House.

In 1912, he was appointed Chief Engineer of the Massachusetts Board of Harbor and Land Commissioners, and, later, as Engineer of Waterways and Public Lands. During this period, Mr. Williams represented the State of Massachusetts in connection with the construction of the Cape Cod Canal.

---

[2] *Colgate Alumni News,* April, 1930.

[1] Memoir prepared by Charles R. Gow and James W. Rollins, Jr., Members, Am. Soc. C. E.

He also conducted a survey and reported upon the water and power resources of the State.

In 1920, when the Massachusetts Commission of Public Works was created, Mr. Williams was appointed its Bridge Engineer and remained in that capacity until December 6, 1922. At this time he was selected by Governor Cox as Chairman of the Commission, in which capacity he directed all State highway construction, including the operations of the Registry of Motor Vehicles, and, in addition, supervised the administration of all State-owned harbor facilities and such construction as was connected therewith. In this position he assumed responsible charge of an annual expenditure approximating $15 000 000. His administration of this important office was marked by an utter absence of any public criticism and a pronounced improvement in efficiency of operation. He retired from this office on December 25, 1928, having attained the legal age limit of 70 years.

After his retirement from public office Mr. Williams was engaged by the Commonwealth to advise and represent the State in connection with a legislative measure for the abolition of railroad grade crossings. He was similarly employed in a consulting capacity to advise the Commission to Investigate Pollution of Boston Harbor, as well as a commission appointed to report upon the filling in of a portion of Fort Point Channel and South Bay in Boston.

As a public official Mr. Williams was an ideal representative. He was not only an eminent engineer; he possessed to an unusual degree the attribute of sound common sense and, in addition, displayed rare executive ability, which combination resulted in a high standard of efficiency in the conduct of his office. He won the respect and complete confidence of the many State, County, and Municipal officials with whom he had to deal. He was at his best when appearing before the several legislative committees having jurisdiction over matters affecting his Department. So impressed were they with his judgment that he once said, "No bill affecting my Department which I opposed, ever passed the Legislature during my term of office."

Mr. Williams was a man of unusually attractive personality and friendly persuasiveness. Always a thorough gentleman under every circumstance, he was able to meet and overcome trying situations by virtue of his calmness, poise, patience, and understanding of human nature. His logic was as invariably sound and clear as his arguments were convincing. He was uniformly popular alike with superiors, associates, and subordinates. His judgment was exceptionally keen and extended into a wide variety of fields other than that of his chosen profession.

He was a delightful companion, taking always a loving, living interest in his fellow men and maintaining a broad outlook upon life. His early years had been filled with adventure, and his reminiscences of those experiences recited in his customary modest and lucid style invariably captivated his listeners. Mr. Williams numbered among his intimates men from every walk of life, including many of the leading citizens of his State. He was a most welcome guest at the firesides of his host of friends who loved and sought his companionship.

He was a distinguished member of the Engineering Profession for almost fifty years without a single blemish on his record. His keen powers of analysis permitted him to render wise decisions respecting problems with which often he had enjoyed little previous experience. He possessed the rare faculty of developing the facts clearly and of drawing sound conclusions from them. His opinion was widely sought by his fellow engineers because of this unusual gift.

Mr. Williams maintained constantly a rigid standard of professional ethics. He invariably held himself aloof from controversy and endeavored to harmonize conflicting opinions. He steadfastly refused to deviate from the paths of duty as he saw them, even to favor a personal friend. He stood loyally by his superiors whenever put to the test, and no official was ever embarrassed even in the slightest degree by any action or statement of his. He was an inspiration to all who were fortunate enough to be associated with him in his work, and his example might well be emulated by the younger generation of present-day engineers.

He died suddenly on October 1, 1929. His passing leaves a void in the hearts of many who knew him and who had learned to love him for his personal qualities. The estimate of his oldest friend is,

"A noble soul has lived, stamped its mark on the world, and gone, but its record will live, and the love and admiration we have for him will last beyond life's closing day. For more than sixty years I have been his friend and confidant. During that long period we have never had an unpleasant word. I have never known him to have an ignoble thought or do a wrong act. He was loyal, faithful, and true to himself, his friends, and his duty."

Mr. Williams was married to Anna L. Cornor on April 7, 1883. He is survived by two sons, Thomas W. Williams, Assoc. M. Am. Soc. C. E., of New Bedford, and Harold Williams, of Pelham, N. Y., and two daughters, Mrs. F. W. Oesting, Jr., of New Bedford, and Mrs. James M. Buchanan, of Maplewood, N. J. A sister, Mrs. Edgar R. Lewis, of Prosser, Wash., also survives him.

Mr. Williams was elected a Member of the American Society of Civil Engineers on April 4, 1906.

CARLTON CARPENTER WITT, M. Am. Soc. C. E.[1]

Died December 17, 1930

Carlton Carpenter Witt was born in Indianapolis, Ind., on December 15, 1870, the son of Bennett F. and Lucy (Carpenter) Witt. His father and mother continued to reside in Indianapolis until their death some years ago. Mr. Witt's childhood and youth were spent in that city. He entered Purdue University, Lafayette, Ind., and was graduated therefrom in 1892 with the degree of Bachelor of Science in Mechanical Engineering. Prior to his graduation from Purdue, he had become interested in mechanical engineering work and, in 1895, he entered the service of the Lassig Bridge and Iron Works of Chicago, Ill., serving, successively, as Designer, Draftsman, and Chief Draftsman.

---

[1] Memoir prepared by C. H. Spencer, Superv. Engr., Bureau of Valuation, Interstate Commerce Commission, Washington, D. C.

In 1898, Mr. Witt entered the employ of the Chicago and Northwestern Railway Company as Chief Draftsman and, later, as Assistant Engineer on Construction, Bridge Department, and continued in that service until November, 1907.

In 1907, he was appointed Chief Engineer of the Railroad Commission of South Dakota and directed the preparation of a complete valuation of railroads and other utilities in that State. In 1911, he was appointed Chief Engineer of the Public Utilities Commission of Kansas, and performed similar duties with that Commission. The work done by Mr. Witt in connection with the two State Commissions involved the complete appraisal of more than 16 000 miles of railroad, in addition to a large number of electric light, telephone, and gas properties.

Immediately after the enactment by Congress in 1913 of the Transportation Act, requiring a Federal valuation of railroads throughout the United States, Mr. Witt was appointed District Engineer of the Western District for the Bureau of Valuation, Interstate Commerce Commission. He became the immediate Assistant of the Member of the Engineering Board for that District, in charge of field and office work for this stupendous undertaking. In 1919, on the retirement of the latter, Mr. Witt was promoted to the position of Member of the Engineering Board and continued as such until November, 1921.

In 1921, on the completion of the field work, covering approximately 50 000 miles of railroad in the Western District, the five districts of the United States were consolidated, with headquarters in Washington, D. C. Mr. Witt, with a considerable number of his force, was transferred to that city where he assumed the position of Assistant Supervising Engineer of the Bureau of Valuation, charged with the completion of the valuations of all railroads in the United States.

Coming into his new surroundings, he quickly won the regard and esteem of all his fellow employees who soon recognized in him a man thoroughly versed in the problems facing a valuation engineer. His gentlemanly qualities and the spirit of fairness accorded those who came in contact with him, won him a host of friends. Railroad officials quickly recognized his integrity and fair dealing, as well as his sure knowledge and acquaintance with all problems with which he had to deal.

In 1924, on the death of Howard M. Jones, M. Am. Soc. C. E., then Supervising Engineer, Mr. Witt was promoted to that position in entire charge of the engineering work connected with the valuation of 250 000 miles of railroad, the primary valuations of which had been brought to a practical completion just prior to his death.

By Mr. Witt's death, the Interstate Commerce Commission lost one of its ablest and most valued officials, and the Profession an engineer of unexcelled ability, known and esteemed throughout the railroad world.

Mr. Witt was a member of the American Railway Bridge and Building Association, a Director of the Washington Society of Engineers, and a member of the Cosmos Club, in the City of Washington.

He was married on February 10, 1921, to Helen R. Creasey, of Kansas City, Mo., who, with two children by a former wife, survives him.

Mr. Witt was elected a Member of the American Society of Civil Engineers on April 6, 1909.

MORGAN EDWARD YEATMAN, M. Am. Soc. C. E.[1]

Died November 17, 1929

Morgan Edward Yeatman was born at East Sheen, Surrey, England, on August 8, 1851, the son of Morgan and Mary C. L. (Penrhyn) Yeatman. He was educated at Marlborough, graduated with honors in Mathematics and Classics at Trinity College, Cambridge, became Wrangler in 1874, and Master of Arts in 1877.

Mr. Yeatman was articled in 1874 to George Fosbery Lyster, Chief Engineer to the Mersey Docks and Harbour Board, and was engaged on extensive new dock and harbor works for that Board, including the preparation of the first plans for the Liverpool Overhead Railway from 1877 to 1881.

He then came to the United States and served until 1884 as Assistant Engineer on the construction of the Pittsburgh, McKeesport, and Youghiogheny Railroad, under the late Jonathan Wainwright, M. Am. Soc. C. E., Chief Engineer. Mr. Yeatman was engaged on the design and construction of many of the bridges on that road.

From 1884 to 1885, he was Principal Assistant Engineer (Maintenance) on the Pittsburgh and Lake Erie Railroad (Lessees of the Pittsburgh, McKeesport, and Youghiogheny Railroad). In 1886, with Mr. Wainwright, he entered into private practice as Consulting and Constructing Engineer, with headquarters in Pittsburgh, Pa. In this capacity he was engaged until 1888 on various bridges and railroads, his experience including bridge detailing for the Keystone Bridge Works and the Iron City Bridge Works (C. J. Schultz and Company).

From 1889 to 1896, Mr. Yeatman was Principal Assistant Engineer, Maintenance of Way, on the Norfolk and Western Railroad, with headquarters at Roanoke, Va., in charge of bridge work on 1 500 miles of line. This work included the renewal of a large number of the older bridges. He compiled the first Standard Bridge Specification for this railroad.

From 1898 to 1908 he was on the staff of S. Pearson and Son, on the design of new bridges for the Tehuantepec National Railroad, in Mexico, and the Arica-La Paz Railway in Chile. He also designed the structural steelwork for the Mexican Ports of Coatzacoalcos and Salina Cruz, including wharves, quay walls, breakwaters, and other harbor works, drainage works, and water supply.

---

[1] Memoir prepared by S. Alfred Frech, Cons. Civ. Engr., London, S. W. 1, England.

In 1908, Mr. Yeatman began the private practice of engineering in England which practice he continued in Westminster until 1915. He was a Member of the Institution of Civil Engineers. During the World War he gave his services to the Mechanical Warfare Department, and was engaged chiefly in the design of "tanks", until the Armistice was signed in 1918.

In 1888, at Pittsburgh, he was married to Blanche D. Fullerton, and is survived by his widow and three children—two sons, Morgan John, who is Lieutenant-Commander in the Royal Navy, and Edward Stanley; and one daughter, Ellinor (Mrs. C. E. May).

He was a man of many hobbies; he was fond of sailing and was an exceptionally fine musician. By his death, which occurred at his residence at Hill House, Buckingham, on November 17, 1929, after a short illness, at the advanced age of 78 years, the Engineering Profession loses a man of keen intellect, a great mathematician, and bridge expert.

Mr. Yeatman was elected a Member of the American Society of Civil Engineers on May 14, 1892.

SAMUEL McCAIN YOUNG, M. Am. Soc. C. E.[1]

DIED OCTOBER 28, 1930

Samuel McCain Young, the son of Dr. John W. Young and Mary Louise (McCain) Young, was born in Carroll County, Mississippi, on December 14, 1873. He was graduated from Washington and Lee University in 1898 with the degree of Civil Engineer.

After his graduation Mr. Young's professional career for a time took him into many fields of effort. He was on United States Deep Waterways Surveys in New York State, on flood protection work on the Mississippi River, and on preliminary and location surveys for railroads, serving at different times the Choctaw, Oklahoma, and Gulf, the Chicago, Rock Island, and Pacific, and the New Orleans Great Northern Railway Companies. He constructed water-works and electric light plants, and electric railways. He opened and operated a rock quarry, and had activities in still other branches, including general practice as a Consulting Engineer, and military services as Captain of Engineers during the World War.

His outstanding professional work, however, was in the line of port development. Mr. Young was Chief Engineer of the Board of Commissioners of the Port of New Orleans, La., for two years prior to the war and again occupied that position from 1921 until his death. He displayed infinite ingenuity and ability in coping with the many problems with which he was confronted in this great work, and his part in the development of the Port of New Orleans was such that it entitles him to a place of special honor in the annals of that Port.

Mr. Young always showed a lively interest in civic affairs, and his services were available at any time to aid in community development. He expended a great deal of time in the study of the traffic and transportation problems of the City of New Orleans, formulating plans for a Union Station layout, and trying to harmonize public opinion thereon. He was an active member of the New Orleans Planning and Zoning Commission, and performed many other duties of value to the city.

---

[1] Memoir prepared by J. F. Coleman, Past-President, Am. Soc. C. E., and W. T. Hogg, M. Am. Soc. C. E.

Mr. Young was elected an Alumni Member of the Washington and Lee Chapter of Phi Beta Kappa in June, 1930. He was a member of the Delta Tau Delta Fraternity, a Mason, and a Knight Templar. He was also a member and a Past-President of the Louisiana Engineering Society and was always active in its affairs. He was a Past-President of the Louisiana Section of the Society, in which he was much interested.

Mr. Young was a man among men, straightforward, honorable, and true. He was a skillful and accomplished engineer, and a highly efficient executive. All who came in contact with him were impressed with his sincerity and his love for his fellow man, and, in consequence, few men have possessed more friends than he. He was a good son and brother, a good husband, a good friend, a good citizen.

He was married in 1923 to Katherine Harrison, and is survived by his widow, by three sisters, one brother, and by his father who was a distinguished member of Forrest's Cavalry during the Civil War.

Mr. Young was elected an Associate Member of the American Society of Civil Engineers on June 7, 1905, and a Member on September 6, 1910.

RAYMOND ARDEN EDWARDS, Assoc. M. Am. Soc. C. E.[1]

Died July 13, 1930

Raymond Arden Edwards was born in San Francisco, Calif., on July 30, 1886. His father, Harry B. Edwards, was of English descent, the son of one of the Governors of the Penal Colony of Australia. His mother, Alice (Doud) Edwards, was of Scotch ancestry, the daughter of Aaron Doud, a well-known pioneer of San Francisco, Calif.

Mr. Edwards obtained his early schooling in Los Gatos, Calif. He had not quite finished High School when his father died and at the age of seventeen he was forced to leave school and go to work. He began his business career as a hardware store clerk, but he gave it up after six months and took a position as office boy with the Pacific Mail Steamship Company. His early ambition was to become a civil engineer, and he devoted all his spare time to poring over engineering books. Mr. A. B. Southard, a well-known San Francisco engineer, encouraged and helped Mr. Edwards in his studies. He gave him his start in engineering by employing him as Chainman on the San José and San Francisco Electric Railroad in 1905 and from this position he rapidly advanced to that of Leveler and Transitman.

From July, 1906, to August, 1907, under the direction of Mr. Southard, he was in charge of field work and the construction of a mining flume in the

---

[1] Memoir prepared by R. W. Olmsted, Assoc. M. Am. Soc. C. E.

Yuba River Canyon, a concrete building, sea walls, a pier, and water supply works at Clear Lake, Calif.

While gaining practical experience in construction, Mr. Edwards, with a determination that was characteristic of everything he undertook, pursued his engineering studies by attending night school. He developed a natural ability to organize and superintend construction projects and from August, 1907, to July, 1912, he had charge of several important works of engineering in Northern California. During this period his list of accomplishments includes sub-division and street improvement work, the construction of a seven-story steel frame and concrete building, highway and dam construction, irrigation works, and a large concrete railroad roundhouse.

The field of irrigation engineering attracted him and in July, 1912, he became Superintendent for the Beard Construction Company on the building of the Goodwin Dam on the Stanislaus River, California. This dam diverts approximately 1 400 cu. ft. per sec. into the canal systems of the South San Joaquin and Oakdale Irrigation Districts, supplying water to about 150 000 acres of valley lands.

In April, 1913, Mr. Edwards was made Chief Engineer of the Modesto Irrigation District, in charge of hydrographic investigations and construction and also of the operation of the extensive canal system delivering water from the Tuolumne River to 83 000 acres of irrigable land. He planned and carried out many important betterments for the irrigation system during his two years as Chief Engineer.

In March, 1915, he joined the United States Reclamation Service as Engineer in charge of field investigations for the preliminary report on the Lower Pitt River Irrigation Project, in California.

In January, 1916, Mr. Edwards became Supervising Engineer for the Foster and Kleiser Company, Outdoor Advertisers, with headquarters in San Francisco. He pioneered in a field that has since attracted a large number of civil engineers. He was responsible for many of the improvements that were made in the outdoor advertising industry, and instrumental in the standardization of this medium throughout the United States.

In July, 1919, he was appointed to be Southern California Manager of the Foster and Kleiser Company, and, under his able direction, the business in this section grew to large proportions and reached a high state of perfection. His executive ability and advanced ideas brought him further recognition and, in July, 1923, he was called to the Main Office in San Francisco to create and manage a Statistical and Research Department; and in July, 1928, he became San Francisco Branch Manager, a position he held at the time of his death.

Mr. Edwards was a very public spirited citizen. He never hesitated to give his time and energy to civic affairs. He was a member of the Planning Commission in San Francisco, a Rotarian, and Secretary of the Club in Los Angeles. He was a Thirty-second Degree Mason, a member of the Young Men's Christian Association Board, and active in Advertising Club work.

His delightful personality and keen sense of humor won for him a host of friends both within and without the profession. He had a keen understanding of human nature. Men who worked under him loved him. He was extremely energetic, almost tireless, and always ready to lend a helping hand to others.

For recreation, Mr. Edwards turned to hunting and fishing. He had a keen liking for the outdoor life and never missed an opportunity to spend a week-end or vacation in the mountains. He had purchased a large tract of mountainous land in Napa County, California, and had built a hunting lodge where he could take his family and friends for outdoor recreation.

Death came to Mr. Edwards very suddenly while at his mountain lodge. A long tramp through the hills on an extremely hot day was too much even for his tremendous strength and healthy constitution, and he died of heat prostration. His passing, in the prime of life, was a great loss and shock to his family and to his many friends.

He was married on April 28, 1912, to Adelaide May Roberts, of San José, Calif., who, with two children, Jane and Harry Wilson, survives him.

Mr. Edwards was elected a Junior of the American Society of Civil Engineers on June 24, 1914, and an Associate Member on November 28, 1916.

WILLIAM JAMES FULTON, Assoc. M. Am. Soc. C. E.[1]

DIED AUGUST 21, 1930

William James Fulton was born on July 13, 1889, at Waddington, N. Y., the son of John and Isabel Fulton. He attended Cornell University at Ithaca, N. Y., from which he was graduated as a Civil Engineer in 1912.

Mr. Fulton's first position after his graduation was with the American Bridge Company, at the Brooklyn, N. Y., Plant, as a Draftsman and Designer on building work. After a year he went to Syracuse, N. Y., where he was employed by the Archbold-Brady Company as Draftsman and Designer on transmission towers. In 1916, he joined the engineering forces of the Sement-Solvay Company, also of Syracuse, as a Designer of buildings, bins, and general structural work.

On June 27, 1917, shortly after the entry of the United States into the World War, Mr. Fulton enlisted in the Army at Fort Slocum, New York. He was soon appointed Corporal in Battery E of the 16th Regiment of Field Artillery at Camp Robinson, Wisconsin, and, in September, was transferred as Sergeant to Fort Oglethorpe, Georgia. He received his commission as Second Lieutenant on December 6, 1917, and was appointed to Company B, 327th Brigade, Tank Corps. On March 9, 1918, he went overseas, where he served with credit and distinction for eleven months. He received his discharge as First Lieutenant on May 19, 1919, at Camp Meade, Maryland.

In October, 1919, Mr. Fulton went with the Montana State Highway Department, at Helena, as Squad Boss in charge of highway design. In a short time he was promoted to the position of Assistant Engineer, with the additional duties of making preliminary inspections, determining grades, alignment, structure type, and location on all highway projects.

---

[1] Memoir prepared by Fred L. Klein, Esq., San Francisco, Calif.

In October, 1921, he was appointed Designing Engineer with the New Mexico Highway Commission, at Santa Fé. At that time the State was expanding its highway program and Mr. Fulton's executive ability was soon recognized by his superiors. He was given the task of re-organizing and building up the personnel of the drafting-room and had charge of the preparation of all road plans, route maps, and estimates. The State had no bridge engineer, nearly all the bridges being of the standard trestle type. Progress in highway design showed that heavier and more permanent structures were needed, and, in the summer of 1921, Mr. Fulton was promoted to serve as State Bridge Engineer. He was given the task of creating a Bridge Department, and with his characteristic energy he worked night and day, making long trips about the State to inspect bridge sites and to determine the type best suited for the location. He soon built up a smooth-running organization which testified to his ability.

He left Santa Fé in July, 1924, and engaged in private practice, specializing in highways, bridges, and grade separations.

In June, 1926, Mr. Fulton was appointed Bridge and Office Engineer for Caddo Parish, at Shreveport, La., having charge of all highway and bridge projects.

His untimely end, at the hands of a disgruntled draftsman, whom it had been necessary to discharge a month previously, cut short a career which promised great success in the engineering field. He had a high sense of responsibility and devotion to duty. He was energetic, resourceful, and had a marked degree of initiative, and all work which was entrusted to him was carried out with painstaking care. He was devoted to his family and left a place in their hearts which will never be filled.

On October 4, 1921, Mr. Fulton was married to Ann Kanthack, of Bluffton, Minn. He is survived by his widow; a son, Billy; his father, John Fulton; and two sisters, Edith, of Waddington, and Mrs. R. E. Taylor, of Fortuna, N. Dak.

Mr. Fulton was elected an Associate Member of the American Society of Civil Engineers on April 7, 1924.

ARTHUR BATES GOODWIN, Assoc. M. Am. Soc. C. E.[1]

Died September 21, 1930

Arthur Bates Goodwin was born in New York, N. Y., on August 10, 1884, the son of Robert and Ella (Bates) Goodwin. He received his education at the Dobbs Ferry, N. Y., Preparatory School, and at Columbia University, New York City.

In 1905, Mr. Goodwin entered the employ of the Board of Water Supply of the City of New York in connection with the construction of the City's waterworks.

---

[1] Memoir prepared by Harry W. Levy, Prin. Asst. Engr., Office of Commr. of Public Works, Borough of Manhattan, New York, N. Y.

From 1907 to the date of his death he was with the Department of Public Works, Borough of Manhattan, New York City, where he advanced along engineering lines through industriousness and capability to the position of responsibility which he occupied at the time of his death. For many years, Mr. Goodwin had responsible charge of the repair gangs in the maintenance of the streets of the Borough of Manhattan.

In 1929, he was assigned as Engineer in the field to supervise and control the work of paving, repaving, and regulating streets, he also was in charge of the construction of new sewers, and the reconstruction and alteration of old ones.

When the United States entered the World War, Mr. Goodwin was among the first to volunteer his services. His war record is as follows: He accepted an appointment as First Lieutenant, Engineer Officers Reserve Corps, in July, 1917, and was assigned to active service at the Engineer Training Camp, American University, Washington, D. C., where he was placed on duty with the 305th Engineers. He was then transferred to Washington Barracks, Washington, D. C., where he organized the 464th Engineers' Pontoon Train, and was promoted to the rank of Captain. He left the United States with his organization in July, 1918, and arrived in France on August 4, 1918. He was ordered to Jaulgonne (Aisne) for the purpose of salvaging all German pontoon equipage found in the valley of the Marne River in that area. Subsequently, he served in the St. Mihiel and Meuse-Argonne offensives, returning to the United States in April, 1919; he was honorably discharged on June 17, 1919. He accepted a commission as Captain, Engineer Officers' Reserve Corps, on November 20, 1919, and later was promoted to the rank of Major, which he held at the time of his death.

The passing of Major Goodwin is a distinct loss to the Office of the Borough President of Manhattan. He was a capable and efficient worker, and possessed courage and vision to "see a thing through". One of his last acts was the supervision of the repaving of Fifth Avenue from 42d to 23d Streets (one of the most prominent and heavily traveled thoroughfares in the Borough of Manhattan). He received special commendation for the manner in which this work was conducted, not only from his superiors in office, but from civic associations and the general public as well.

Taken ill with appendicitis, an operation revealed the fact that the appendix had been ruptured, with the result that Major Goodwin died from peritonitis on September 21, 1930. He was buried with full military honors in Sleepy Hollow Cemetery, at Tarrytown, N. Y. He is survived by his widow, Louise Du Pre Goodwin, his mother, Mrs. Ella B. Goodwin, and his sister, Mrs. Robert Curry.

At the time of his death, Major Goodwin was Past Commander of the President of the Borough of Manhattan Post No. 84, American Legion, which he organized in 1926, and in which he had served as Commander for the first three years of its existence. He was also a member of Westchester Lodge, F. and A. M., and the Supreme Council of the Scottish Rite.

Major Goodwin was elected an Associate Member of the American Society of Civil Engineers on July 6, 1920.

CHARLES ELLETT HART, Assoc. M. Am. Soc. C. E.[1]

DIED JANUARY 3, 1929

Charles Ellett Hart was born in Bay City, Mich., on June 16, 1888, the son of Charles Matthew and Lottie Elizabeth (Kline) Hart. After completing his elementary training in the Bay City public schools, he attended the University of Michigan, and was graduated from the Engineering Department in 1911. During the summer of 1910 he acted as Draftsman for the American Bridge Company at Ambridge, Pa.

In July, 1911, Mr. Hart entered the employ of the Truscon Steel Company in Detroit, Mich. and his entire professional career was with this Company from that time until his death in January, 1929. Starting in the Drafting-room he advanced through the stages of detailing, checking, estimating, and designing, to the head of the Engineering Department. Later, he was transferred to the Sales Department.

Mr. Hart's career lacked the striking or spectacular accomplishments so often met with by the engineer in outside activities. His was rather one of steady, quiet progress. Personally, he was not the aggressive type—rather modest and retiring, but sincere and dependable and very well liked by those with whom he came in contact.

Mr. Hart was elected an Associate Member of the American Society of Civil Engineers on June 1, 1920.

ROBERT GRIER HEMPHILL, Assoc. M. Am. Soc. C. E.[2]

DIED JUNE 19, 1930

Robert Grier Hemphill, the second son of the late Gen. Robert R. Hemphill and Eugenia (Taylor) Hemphill, was born on February 6, 1885, at the family residence in Abbeville, S. C.

He attended the graded school at Abbeville and then entered Charleston College from which he was graduated with high honors in 1903 at the age of eighteen. After teaching school for a session at Monroe, Ga., he entered Clemson College with the object of taking a course in Civil Engineering. Failing health, however, forced him to abandon the course before the first year was completed. A period of two years was spent chiefly at Saranac Lake, New York, in efforts to regain his health, and, at its close, he was advised by his physicians to find employment if possible in one of the Rocky Mountain States.

There happened to be at that time a subordinate position open in Colorado under Elwood Mead, M. Am. Soc. C. E., who was then in charge of the "Irrigation Investigations" of the United States Department of Agriculture, and in June, 1906, Mr. Hemphill received the appointment.

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

[2] Memoir prepared by Samuel Fortier, M. Am. Soc. C. E.

It was at the base of the Rockies that he laid the foundation of that extensive knowledge, which he later acquired, of the intricate relationship of plants to water. The trees of the orchard and the plants of the field became to him members of his household. To some, he gave large quantities of water, to others, small or medium amounts. If one cultural method seemed to be faulty, he tried another, always watching with critical eye and measured results the effect on the plants.

As the years passed, the scope of his work broadened, and larger problems engrossed his attention. Clustering around the old Union Colony which Horace Greeley founded in 1870, amid cacti and jack-rabbits, were to be found the most productive lands of Colorado, the crop returns from which yielded $25 000 000, or more, annually. The wealth derived from these soil products depended almost entirely on one mountain stream—the Cache la Poudre. Mr. Hemphill and his associates undertook to determine how this valuable water supply was diverted or stored, distributed, and used, with a view to the further definition and protection of water rights to individuals and communities, and the betterment of irrigation practice. In Mr. Hemphill's report covering this work, entitled "Irrigation in Northern Colorado", which is a classic of its kind, he traces the water from the time it leaves its rocky canyon through many canals, laterals, and farmers' ditches, until it is transpired by the foliage of plants. Carrying capacities, seepage losses, storage, exchange of water, water rights, distribution, and use, are all considered. No statement is made that is not supported by fact and all is presented in simple, clear, and expressive English.

If Mr. Hemphill had done nothing more than prepare this report he would have been entitled to the plaudits of the rural population of the Rocky Mountain region, but the solution of still larger water problems was demanded. This time it was the South Platte River, of which the Cache la Poudre was but a tributary. What is the return flow of this river, and how can it be made to serve best the interests of farming communities? These were the problems to be solved. Here, again, thousands of water measurements had to be made, the results compiled, and a report prepared by Mr. Hemphill's superior, Ralph L. Parshall, Assoc. M. Am. Soc. C. E.

With the completion of this and other irrigation investigations in Colorado, Mr. Hemphill's duties and responsibilities were enlarged by receiving an appointment on July 1, 1920, as Resident Irrigation Engineer for Western Texas. In this new field, with its different climatic and plant conditions, he was obliged to study once more the water requirement of plants, and, with this knowledge as a nucleus, to extend the activities of his Bureau to streams, pumping plants, irrigation systems, irrigation requirements, and the welfare of farming communities in general.

From 1925 his major work had consisted in a study of river silt in Texas streams and its relation to the life of reservoirs.

From 1907 to 1925, it was the privilege of the writer to help plan Mr. Hemphill's work, direct its operation, and supervise the results. In this long association the writer early learned to place implicit confidence in his honesty, ability, and painstaking efforts. If working eight hours a day did not suffice, ten or twelve hours would be devoted to the task, and when it was completed reliance could be placed on the facts submitted. These qualities of industry, perseverance, and accuracy, combined with a cheerful disposition and reticence to allude to his physical disability, endeared him to all his co-workers.

Mr. Hemphill was married to Jessie Culver, at Canyon City, Colo., on August 3, 1910, but after six years of happy married life, Mrs. Hemphill succumbed to an attack of influenza. They had no children.

Mr. Hemphill was elected an Associate Member of the American Society of Civil Engineers on October 15, 1923. He was also a member of the American Society of Agricultural Engineers.

EDWARD BAXTER HILL, Assoc. M. Am. Soc. C. E.[1]

DIED APRIL 26, 1930

Edward Baxter Hill was born in Syracuse, N. Y., on August 9, 1890. He was the son of Bella S. and the late William Ryan Hill, M. Am. Soc. C. E., who was serving as a Director of the Society at the time of his death on June 16, 1918.

Edward Baxter Hill received his early education in the public schools of Syracuse and Brooklyn, N. Y. After the family moved to Albany, N. Y., in 1907, he attended the Albany Academy and, later, the Troy Academy. After his graduation from the Troy Academy in 1913, he attended Rensselaer Polytechnic Institute for two years.

He was first employed by the United States Geological Survey as a Rodman in the summer of 1912, and he was re-employed in the same capacity, when not attending school, until 1917. On May 8, 1917, he was appointed a Field Aide, and on June 1, 1919, Junior Topographer. He resigned from the Geological Survey in December of the same year, but was reinstated in the following June. He served in various grades until December 8, 1927, when as Assistant Engineer he was transferred to the United States War Department.

During his employment by the War Department, Mr. Hill had his headquarters at the United States Engineer Office, Rock Island, Ill., where he was engaged in the compilation of maps of the Mississippi River from St. Paul, Minn., to St. Louis, Mo.

His work for the War Department having been completed, Mr. Hill was transferred back to the U. S. Geological Survey on October 26, 1928. After his return to the Survey, he was engaged in map compilation, geodetic computations, and topographic surveys. He was engaged on a topographic survey in Kentucky when he was stricken with the illness that resulted in his death.

Mr. Hill was careful, conscientious, and able in his work as an engineer, to which he devoted himself with persistence and industry. By his fairness, kindness, and generosity, he won the friendship and affection of his associates and of those with whom he came in contact. He was a splendid example of American manhood and his fondness for and devotion to his home and family were notable traits of his character.

---

[1] Memoir compiled by J. G. Staack, M. Am. Soc. C. E.

In December, 1917, Mr. Hill was married to Leila Hart, of Georgetown, Del., who, with a son, Edward, survives him.

Mr. Hill was elected an Associate Member of the American Society of Civil Engineers on July 6, 1925. He was also a member of the Washington Society of Engineers.

EDWIN ALLSTON HILL, Assoc. M. Am. Soc. C. E.[1]

Died October 28, 1929

Edwin Allston Hill was born in New York, N. Y., on January 18, 1850, the son of Benjamin Scranton and Elizabeth (Stokes) Jones Hill, who was a lineal descendant of Deputy Governor Jones of the New Haven Colony. He also had two *Mayflower* lines—John Alden and John Howland.

Mr. Hill received the degree of Bachelor of Arts from Yale University in 1875, and the following year was enrolled as a student in the Columbia Law School. His further honorary degrees were, Master of Science and Doctor of Philosophy from George Washington University in 1901 and 1903, respectively, and Master of Arts from Yale in 1902. Mr. Hill was admitted to the New York Bar on May 25, 1878, and to the Connecticut Bar on October 28, 1879, on examination.

In 1877, he was employed with the legal firm of Anderson and Man, in New York, and in the fall of that year went to New Haven, Conn., on legal work for this firm in connection with the real estate of the Boston and Air Line Railway, formerly a Division of the New York, New Haven, and Hartford Railroad. He continued with the Boston and Air Line Railway Company, as Assistant Attorney, Right-of-Way and Claim Agent, and Assistant Superintendent until January, 1883, at which time he became engaged in surveys in Southern Connecticut for the Hartford and Harlem Railroad Company.

From 1885 to 1888 he served as Right-of-Way Agent and Chief Engineer for the Indianapolis, Decatur, and Springfield Railway Company, with headquarters at Indianapolis, Ind.

In the summer of 1888, Mr. Hill became Real Estate Agent and Tax and Insurance Agent for the Cleveland, Cincinnati, Chicago, and St. Louis Railroad Company, and, later, Assistant Engineer, establishing his home at Fernbank, Ohio. From 1893 to 1894 he served as Railway Expert to the banking house of Hatch and Foote, in New York, which firm he represented in Illinois as Assistant Receiver of the Jacksonville-Southeastern Railways System during 1893 and 1894.

In 1894, Mr. Hill returned to New Haven where he remained until December, 1895. He then went to Washington, D. C., as Private Secretary to the Commissioner of Patents, John S. Seymour, and the two following Commissioners, Benjamin Butterworth and Charles H. Duell. On December 3, 1898, he was assigned to duty in the Classification Division, promoted to Third

---

[1] Memoir compiled from information on file at the Headquarters of the Society.

Assistant Examiner on August 28, 1900, Second Examiner, October 2, 1903, and First Examiner on June 28, 1907. He was admitted to the Bar of the Supreme Court of the District of Columbia on February 14, 1903, and to the Court of Appeals of the District of Columbia on February 24, 1903. He was duly admitted and qualified as an Attorney and Counsellor of the Supreme Court of the United States on May 15, 1911. He was also registered to practice as an Attorney before the United States Patent Office on September 9, 1926. While he was in the Classification Division he designed the system of Indexing Chemical Literature and Patents adopted by the United States Patent Office and, later, by the American Chemical Society.[2]

Concurrently with Mr. Hill's legal work in Washington, he was actively engaged on the faculty of George Washington University. He was Instructor and Assistant Professor of Chemistry for evening classes from 1901 to 1926, at which time he became Professor of Chemistry in both night and day schools until 1929. He was elected Professor Emeritus by the Faculty the morning before he died, but his election was too late to be confirmed by the Board.

Professor Hill was always interested in chemical and microscopical matters, even when engaged in engineering work. He also spent much time in genealogical research, and a list of his published literary papers embraces, not only civil engineering, but genealogy, history, chemistry, and other sciences as well. He was also exceedingly interested in music.

He was a member of the following clubs and organizations: The Western Society of Engineers; the Engineering Societies of Connecticut and Illinois, and the Cincinnati, Ohio, and Washington, D. C., Societies of Engineers; a member of the Board of the Washington Chapter of the American Chemical Society; the American Association for the Advancement of Science; the Postal and Washington Microscopical Societies; the State Societies of Ohio, New York, Connecticut, and Vermont; the Alumni Societies of Yale, Columbia, and George Washington Universities; a member of the Board of the District of Columbia Choral Society; the Patent Office Society; the National Genealogical Society; the Connecticut and District of Columbia Societies of Mayflower Descendants, having served as Historian for the former from 1897 until his death, his last act having been to sign his Thirty-second Report, and Secretary and Charter Member of the latter; Vice-President, Registrar, and member of the Board of the District of Columbia Society of Sons of the American Revolution; a member of the Board of the District of Columbia Society of Colonial Wars; the Order of Washington; the Wyoming Club, Jacksonville, Ill.; the Sangamo Club, Springfield, Ill.; the Home Club, District of Columbia; Delta Kappa Fraternity, at Yale, and Alpha Xi Sigma, at George Washington University; also, a member of the Inaugural Committees at the inaugurations of Presidents McKinley, Roosevelt, Taft, Wilson, and Harding; member of the Standing Committee on Street Decorations, and of various Reception Committees for most of the unveilings of statues, in Washington for the last twenty years.

---

[2] See Papers 30, 32, 33, and 34; H. R. Doc. No. 1110, 63d Cong., 3d Session, pp. 599 to 618; also, Condon and Patterson's Guide to the Literature of Chemistry, pp. 191–192.

Professor Hill was married on June 18, 1884, at New Haven, to Ida M., the daughter of Alonzo F. Wood, who survives him.

As a tribute to Professor Hill's memory, the following is quoted from the *Parish Bulletin* of All Souls Protestant Episcopal Church, in Washington, of which he was a member:

> "A scholar of distinction, unostentatious and simple in his ways, he sang in our choir from its early days with a faithfulness and joy that were an inspiration to minister, choir, and people. The mellowness of his years and the youthfulness of his heart were two of the signs of the reality of his Christian experience. Withdrawn from our immediate vision, we know that the tie that binds us to him and him to us is not broken."

Professor Hill was elected an Associate Member of the American Society of Civil Engineers on May 3, 1893.

CORNELIUS JOSEPH O'CONNOR, Assoc. M. Am. Soc. C. E.[1]

Died January 29, 1929

Cornelius Joseph O'Connor, the youngest of three sons of Cornelius and Alice (Gerarty) O'Connor, was born in New York, N. Y., on January 26, 1876. His early education was obtained in the public schools of his native city, where he also prepared for entrance to the College of the City of New York in which he was enrolled as a student from 1892 to 1896. Subsequently, in 1897, he attended extension courses at Columbia University.

In 1896, following the completion of his studies at the College of the City of New York, Mr. O'Connor entered the engineering service of New York City, and remained in the public service almost without interruption to the time of his death. Starting as Sounder, and then as Axeman, in the Bureau of Street Improvements, he was soon transferred to the Department of Sewers in the Borough of The Bronx. He held, successively, the positions of Rodman, Leveler, Transitman, and Assistant Engineer during the twelve years spent in this Department. His work, during this period, covered both preliminary surveys and supervision of construction of sewerage works in a rapidly growing section of the city.

In July, 1910, Mr. O'Connor was appointed Assistant Engineer with the New York Board of Water Supply. This Board was then engaged on the development of the Catskill Water Supply System, consisting of the construction of reservoirs, aqueducts, and arteries of distribution. Mr. O'Connor was assigned to the City Aqueduct Department which was charged with the construction of the City Tunnel, a deep distribution tunnel, 15 to 11 ft. in diameter, extending under three of the Boroughs of the City. He continued on this work for six years, first as Assistant Section Engineer and, later, as Section Engineer on the northerly portion of the work.

On the completion of the City Tunnel, Mr. O'Connor was transferred to the Reservoir Department of the Board of Water Supply under the supervision of which the development of Schoharie Creek, intended to augment the supply from the completed Esopus Creek development, was being started. For somewhat more than a year, he was engaged on riparian surveys on Schoharie Creek. Then, with the commencement of work on the Shandaken Tunnel, an 18-mile aqueduct for the transportation of the waters of Schoharie Creek through the divide to the Esopus water-shed, he was assigned as Senior Section Engineer on its construction, first with the Prattsville Division and, later, with the Allaben Division.

---

[1] Memoir prepared by Roger W. Armstrong and James A. Guttridge, Members, Am. Soc. C. E.

In December, 1921, before the completion of the Shandaken Tunnel, Mr. O'Connor was appointed Division Engineer and assigned to the Southern Division of the Aqueduct Department. This Department had charge of the completion of the steel pipe siphons of the Catskill Aqueduct, of which only one-third of the final installation had been included in the original construction. In this position he supervised the construction of seven siphons consisting of double lines of steel pipes varying from 7 ft. 10 in. to 11 ft. 3 in. in diameter, with concrete envelopes and mortar linings and having an aggregate length of 13 700 ft.

This work was completed in 1926, and Mr. O'Connor then left the Board of Water Supply to accept a position with the Board of Transportation as Adjuster of Claims on subway construction. He remained with this Board for two years reporting directly to the Deputy Chief Engineer.

In 1928, the Board of Water Supply started work on City Tunnel No. 2, a deep distribution tunnel similar to, but larger than, the first. When this project was started, Mr. O'Connor returned to the Board of Water Supply as Division Engineer in charge of the Queens Division. The work of this Division included about 4½ miles of deep pressure tunnel, 17 ft. in diameter, with four distribution shafts and their appurtenant structures. This position he held at the time of his death on January 29, 1929.

Mr. O'Connor was of a genial and sociable disposition and had acquired a large circle of acquaintances and friends. His good nature and pleasing personality, combined with his devotion to duty and fairness in all his dealings, made his business relations extremely pleasant and won for him the confidence and respect of all with whom he had intercourse.

On April 20, 1927, he was married to Ella L. Driscoll, a resident of New York City, and thereafter made his home in Bronxville, N. Y.

Mr. O'Connor was elected a Junior of the American Society of Civil Engineers on March 1, 1904, and an Associate Member on June 5, 1907.

WALDO JUSTIN BICKLE PORTER, Assoc. M. Am. Soc. C. E.[1]

DIED FEBRUARY 6, 1930

Waldo Justin Bickle Porter was born at Crawford, Nebr., on May 9, 1895. His mother, Edith M. Porter, died when he was about four years old, after which time he lived with his grandparents, Mr. and Mrs. James Porter, in Moose Jaw, Saskatchewan, Canada. After graduating from the High School there, he attended Iowa State College, at Ames, Iowa, from which he was graduated as a Civil Engineer in 1916.

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

As soon as the United States entered the World War, Mr. Porter went in training at Fort Riley, Kansas, for a commission in the artillery, sailing for France as a Second Lieutenant in Battery C, Eightieth Field Artillery, of the Seventh Division, with which he served until the end of the war.

On his return to the United States, he entered the Department of Public Works of the State of Nebraska; he was employed in this office for about eight years. He was District Engineer with headquarters at Ainsworth, Nebr., when he resigned that position to accept another with the American Paving Corporation of Omaha as Construction Superintendent and Engineer. He had been in charge of that Company's construction work in Iowa, Kansas, Illinois, Oklahoma, Missouri, and Nebraska.

Mr. Porter was living at Plattsmouth, Nebr., at the time of his fatal illness. Three weeks elapsed before he was taken to the hospital in Lincoln, Nebr., where he died from a ruptured appendix on February 6, 1930; he was buried in Wyuka Cemetery on Sunday, February 9.

He has left a place in the hearts of his relatives, friends, and business associates that never can be filled. He was by nature somewhat quiet, reserved, and conservative in manner, but extremely courteous and considerate in his treatment of others. Naturally of a retiring disposition, his ability and personality were known only to those with whom he was associated, but by them he was held in the highest esteem. He was a member of the Masonic Order, and his religious affiliations were with the Baptist Church. He enjoyed his profession fully, and always took a keen delight in any unusual problems, no matter how difficult or tedious.

On May 8, 1926, Mr. Porter was married to Jean Karnes, of Lincoln. He is survived by his widow, his parents, Mr. and Mrs. J. E. Porter, and a brother, George, of Crawford, Nebr., and a sister, Mrs. Caroline Barnes, of March Field, Calif.

Mr. Porter was elected an Associate Member of the American Society of Civil Engineers on December 5, 1927.

JOSEPH WARREN ROGERS, Assoc. M. Am. Soc. C. E.[1]

DIED NOVEMBER 21, 1930

Joseph Warren Rogers was born on August 29, 1861, at New Brighton, Staten Island, N. Y. His father, also Joseph Warren Rogers, was a member of the New York Stock Exchange, and his mother was Fanny (Edwards) Rogers, who was descended from Jonathan Edwards and from Governor Bradford of the Plymouth Colony. The Rogers family were early settlers in Connecticut, and later generations lived in Huntington, Long Island, and Hyde

---

[1] Memoir compiled from information supplied by William Hauck, Assoc. M. Am. Soc. C. E., and on file at the Headquarters of the Society.

Park, N. Y. His paternal great-grandfather served in the Revolution as one of the Connecticut Minute Men, and afterward became a famous sea captain.

When Mr. Rogers was an infant, his father bought a country place at Scarborough-on-Hudson, N. Y., and it was the family home for nearly forty years. The boy attended The Holbrook School and was anxious to enter West Point, but his parents disapproved and he prepared for college. He was graduated from the Sheffield Scientific School of Yale University in 1884, with the degree of Bachelor of Philosophy. His deep interest in the Army, however, continued throughout his life.

After his graduation Mr. Rogers was employed on construction by the West Shore Railroad Company, and in office work and inspection by the New York Central Railroad Company, until 1891. From 1891 to 1896 he was with the Cataract Construction Company, Niagara Falls, N. Y., as Assistant to the Resident Engineer in charge of tunnel and wheel-pit construction for a hydro-electric plant. In 1897, he was with the Metropolitan Street Railway Company, of New York, N. Y., as Instrumentman and Computer, and as such was engaged on the construction of an underground trolley system. For the first three months of 1898, he was with the East Jersey Water Company on the layout and construction of the head-race canal and pumping station, at Little Falls, N. J.

During the Spanish-American War Mr. Rogers served as Sergeant with the 12th New York Volunteer Infantry, which was part of the Army of Occupation in Cuba. When his regiment was mustered out of service in April, 1898, he returned to the East Jersey Water Company as Assistant Engineer on the construction of the Great Notch Reservoir. In 1900, he resigned his position with this Company and associated himself with the Pennsylvania Railroad Company in charge of the construction of a part of a new single-track branch line on the Middle Division of the System, including rock cut, masonry for 200-ft., plate girder bridges, timber trestle, culverts, etc. He was also in charge of general field work on an additional two-track road to the main line, from Mill Creek to Mt. Union, Pa. For about ten months he was likewise in charge of line and grade and inspection on the work of doubling the width of a ten-arch masonry bridge, 500 ft. long, on a curve on the main line.

In 1903 Mr. Rogers was employed by the Pittsburgh, Toledo, and Western Railroad Company as Inspector in charge of 1 000 ft. of double-track tunnel. In this position he was responsible for the line and grade, quantity and quality of timber, as well as the excavation and back-filling.

In May, 1904, he left railroad construction work to enter the service of the City of New York. For two years he was in the Borough of Richmond as Leveler and Transitman on the construction of streets and highways. In 1906, he was transferred to the Board of Water Supply, then engaged in developing the Catskill Water System, and was assigned to the Reservoir Department, living first at Brown's Station and, later, at Kingston and West Shokan, N. Y.

Beginning with field work, topographical surveys, and preliminary lines for the Olive Bridge Dam and Dikes, Mr. Rogers served as Instrumentman on the relocation of highways. In 1908, he was promoted to the position of Assistant Engineer, on line and grade of East Dike, and of ditches on swamp drainage.

From 1911 to December, 1914, Mr. Rogers was engaged on the inspection of construction work at Olive Bridge Dam, and in taking measurements by planimeter of the actual capacity of Ashokan Reservoir from detailed topographical sheets. On the completion of this dam in 1914, he was sent to Prattsville, N. Y., in the Schoharie Valley, where his work was the inspection and reporting on boring operations in the Schoharie Valley, in connection with the location of Gilboa Dam and the Shandaken Tunnel. He was also in charge of the construction of a model (half-size) of steps of the proposed Gilboa Dam. In 1920, he was in charge of the relocation of highways surrounding the Schoharie Reservoir. At this time, he moved to Grand Gorge, N. Y.

From 1921 to November, 1924, he was engaged on the construction of Gilboa Dam as Assistant Engineer, as well as on general hydrographic work and preliminary survey for the sewer, at Tannersville, N. Y.

From November, 1924, until his death, Mr. Rogers was Assistant Engineer with the Department of Water Supply, Gas and Electricity in the Borough of The Bronx, New York City. His work was in connection with the maintenance and improvement of the distribution system, including "strip maps" of location. Mr. Rogers died on November 21, 1930, of agranulocytic angina, having been absent from the office only five days.

He was a member of the National Security League, of the Society of American Military Engineers, and of the Protestant Episcopal Church.

He was married on November 29, 1911, to Helen Edwards, of Stockbridge, Mass., who survives him.

His recreation was in vigorous out-door activities. Hunting in the Maine woods was the object of his vacations for years, but his love of Nature took him to the near-by mountains and forests at all seasons. When he was first active in the Catskill Water Supply project, he initiated the young men of the Engineering Corps into the joys of mountain climbing and camping, and of trout fishing in remote mountain streams. Many of these men remember him affectionately as "The Scout".

In 1916, he attended the Citizens Training Camp, at Plattsburg, N. Y., and although he was in his fifty-seventh year when the United States entered the World War, he was greatly disappointed not to be called into service. He did everything in his power to assist the younger engineers in preparing for the Army, establishing a rifle range behind his house in Prattsville, where they practiced after working hours. He supported every phase of Red Cross work undertaken in the village, and his home was a center for its activities. He was given to hospitality and, as a small visitor once said, "He has a good idea of a host!"

The most unassuming of men, he would have been amazed at the love and grief expressed when he died, and at the number of engineers and other friends who came hundreds of miles to pay their last respects.

The following letter was addressed to Mrs. Rogers, by Mr. Rogers' associates in the Department of Water Supply of the City of New York:

"It seems fitting you should know the esteem and affection we in this office, held for Mr. Rogers. He was a gentleman (in the finest sense of the word), always industrious and painstaking with his work. He was known for his fairness and pleasantness towards all with whom he came in contact. We feel that we have lost a valuable asset to this office.

"We, who were associated with Mr. Rogers, wish to sign our names as evidence of our sincerity. We extend to you our sincere sympathy and hope that these few words will help you in your sorrow."

Mr. Rogers was elected an Associate Member of the American Society of Civil Engineers on October 1, 1912.

NICHOLAS ALEXANDER SCRIPKO, Assoc. M. Am. Soc. C. E.[1]

Died September 21, 1930

Nicholas Alexander Scripko was born at Gorboff, Chernigovsky, Russia, on May 7, 1889. He was graduated from the Polytechnic Institute of Petrograd, Russia, in December, 1915, with the degree of Civil Engineer.

Previous to taking up a University course Mr. Scripko was employed by the Russian Government on the Trans-Siberian Railway at Kainsk, Tomsky, as Levelman and Draftsman. After beginning his engineering studies in the Polytechnic Institute, he was still employed by the Russian Government during the summer months in the Agricultural Department, first as Instrumentman and then as District Engineer in charge of five well-boring, surveying, and construction parties.

In 1915, not long after the commencement of the World War, he became an Ammunition Inspector for the Russian War Department at Petrograd, and a few months later was transferred in the same capacity to the Russian Embassy, at Washington, D. C. His duties were to inspect ammunition that was being made in Canada and the United States for the Russian Government.

After the Revolution in Russia, his services being no longer needed, he became Mechanical Draftsman with the Marlin Rockwell Corporation of Norwich, Conn., and then Masonry Inspector with the Erie Railroad Company, in New York, N. Y.

When the White Army in Eastern Siberia began to resume activities against the Soviet in 1919, Mr. Scripko went to Vladivostock, Siberia, in charge of the Department of Small Arms and Machine Guns in a movable artillery repair shop sent to the White Army by the Russian Embassy in Washington. In 1920, after these military operations had quieted down, Mr. Scripko became Chief Engineer of the Nikolajevsky Colliery Company, in Vladivostock.

In February, 1921, he returned to the United States, and having arrived in Oakland, Calif., took some post-graduate work at the Polytechnic College of Engineering in that city, specializing in structural and concrete design. On the completion of these studies, he entered the United States Reclamation Service, District 2047, Colusa, Calif., as Instrumentman, then Chief of Party, and, subsequently, Assistant Engineer on a $2 000 000 reclamation project.

---

[1] Memoir prepared by F. A. Kittredge, M. Am. Soc. C. E.

He then in turn became Assistant City Engineer at Antioch, Calif.; Assistant Engineer with the Sugar Pine Lumber Company, at Fresno, Calif.; Resident Engineer for Hunt Mirk and Company, Incorporated, San Francisco, Calif., in charge of the construction of a power plant for a lumber mill; Inspector for the Western Willite Company, San Francisco, on asphalt pavement work; and, again, Assistant City Engineer at Antioch, in charge of field work on the construction of a reservoir.

In 1926, Mr. Scripko went to Martinez, Calif., as Senior Engineer Structural Draftsman with the Shell Oil Company. In 1927, he became Instrumentman with the California Highway Commission at Redding, Calif.; and, subsequently, Surveyor and Draftsman for the Board of Park Commissioners, San Francisco.

In 1928, he entered the National Park Service, at San Francisco, as Principal Engineering Draftsman, being promoted to Chief Draftsman in 1929, and to Assistant Engineer in January, 1930, which position he held at the time of his sudden death of heart failure in September, 1930.

His work latterly had been in the Sequoia National Park where, as also in the Headquarters Office and among his other numerous friends, his frank, open, and generous nature, and ever-constant cheerfulness in all circumstances made him endeared of all and a companion indeed hard to replace.

Mr. Scripko was a member of the High Sierra Club, the Russian Club, and Golden Gate Lodge of Masons, in San Francisco. In 1926, he was married, at Martinez, to Evelyn Hough, who survives him.

Mr. Scripko was elected an Associate Member of the American Society of Civil Engineers on October 14, 1929.

WILLIAM LAURIE SEAMAN, Assoc. M. Am. Soc. C. E.[1]

Died August 14, 1928

William Laurie Seaman was born at Glen Cove, Long Island, N. Y., on July 26, 1894, the son of William H. Seaman and Margaret (Laurie) Seaman. His ancestors on both sides of the family were among the earliest settlers in the Eastern States and on Long Island, extending back to Capt. John Seaman and Capt. John Underhill, the founders of these families, and on his mother's side to a Scottish great-grandfather. His father's paternal great-grandfather was Elias Hicks, the founder of the Hicksite Branch of the Religious Society of Friends, and William Laurie Seaman was thus born into a Quaker family.

As a boy, Mr. Seaman attended the Friends' Academy, at Locust Valley, Long Island, graduating from there in 1911. He entered Swarthmore College at Swarthmore, Pa., in the autumn of that year, and was graduated in 1915, with the degree of Bachelor of Arts in Civil Engineering. He had been prominent in many activities during his college course; he belonged to the Phi Kappa Psi Fraternity, and to the Senior Honor Society, the Book and Key; had served as an Athletic Manager and President of the College Young Men's Christian Association; and was the President of his Class at graduation. A very conscientious, thoughtful, and kindly disposition made him greatly respected and beloved by both Faculty and friends, and his college career was a splendid foundation for later life.

---

[1] Memoir prepared by William H. Seaman, Esq., Glen Cove, N. Y.

Immediately upon leaving college, Mr. Seaman went to work with the Turner Construction Company in New York, N. Y. Between 1915 and 1917, he acted for that Company as Timekeeper, Material Clerk, and Piling Inspector, on construction work in and near New York City, and in South Carolina as Lumber Inspector and Traffic Manager, and, later, as Steel Foreman and Assistant Superintendent.

Mr. Seaman had enlisted in the Signal Reserve Corps of the Aviation Section of the United States Army, and was called in September, 1917, to the Massachusetts Institute of Technology in Boston, Mass., for preliminary training, and then to Texas for flying instruction, where he was commissioned as Second Lieutenant in November, 1917. From there he was sent to Dorr and Carlstrom Fields in Florida, where he acted as Flying Instructor and official Flight Tester, in the Engineering Department, until December, 1918, his commission as First Lieutenant being on its way to him when the Armistice was signed.

At the end of his war service, Mr. Seaman again entered the employ of the Turner Construction Company, as Assistant to the General Superintendent in Philadelphia, Pa. (1919-1921). During his summers, and in all other intervals of employment, he had worked as Transitman, Draftsman, and Chief of Party, etc., with his father, William H. Seaman, Civil Engineer, in Glen Cove. From September, 1921, to March, 1922, he acted as Division Commercial Survey Engineer for the Bell Telephone Company of Pennsylvania, directing and supervising a commercial survey of Philadelphia.

Mr. Seaman left this work to join a college friend in a tour around the world, making economic studies for journalistic and educational purposes. One of his articles, of interest to engineers, and illustrated with his own camera pictures, on the open coal mine in Fushun, South Manchuria, the thickest coal seam in the world, was printed in *Coal Age,* in April, 1923.

On his return to the United States, Mr. Seaman worked with his father until his departure for Australia, in June, 1924, where he was to be married. In the spring of 1925, he went to Sydney and entered into business with Mr. E. W. Bell, Loss Assessor, as head of the Engineering Department. Here, Mr. Seaman spent three very successful years. He was made a partner in Mr. Bell's firm, entering into the life in Sydney with great zest, joining the Automobile Club, the Aero Club, a Ski-ing Club, the American Club, of which he was almost immediately made Secretary and Treasurer, and the Australian Engineers' Club. In the United States, he previously had been a member of the American Legion, American Flying Club, the Aero Club of America, and the Swarthmore Alumni Association and Men's Club.

Laurie Seaman was always fond of Nature and the out-of-doors and of games, as a recreation. When in Japan, with a party he climbed to the summit of Fujiyama, and during his second winter in Sydney, he joined a few members of his Ski-ing Club, and reached the summit of Mount Kosciusko, in the Australian Alps, although it was his first real experience in mountain climbing on skis. This interest, however, by another year had brought to its close a very promising, intensively lived young life. In the Australian winter of 1928, in August, Mr. Seaman went for a vacation in the snow to Mount Kosciusko. A party went out on skis, and Mr. Seaman and a companion pushed on ahead, making the trip to the summit. On their return, however, they became separated and, in the sudden blizzard which had arisen, both lost their lives. When found, all evidence pointed to the fact that Mr. Seaman had succumbed to exhaustion and sleep, while waiting on the right road for his friend, who had apparently gone far out of the way.

The writer of a memorial[2] speaks of Mr. Seaman thus:

"Of a quiet, retiring disposition, kindly and considerate in his make-up, he was one of Nature's gentlemen. A huge success generally, he possessed a host of friends, who loved him for his sterling worth. Although world travelled, he was not a man of the world, rather a man of the home, a genuine husband and thoughtful father, a stickler for the best things in life. Some fine fellows have departed this world; Laurie, one of them, left it, though still climbing."

Mr. Seaman was married on June 14, 1924, to Christiana Bell, of Sydney, Australia, who survives him, with two sons, Bruce Laurie and Ernest William Laurie.

Mr. Seaman was elected a Junior of the American Society of Civil Engineers on October 14, 1919, and an Associate Member on November 15, 1926.

ERASTUS ROLAND SIMPSON, Assoc. M. Am. Soc. C. E.[1]

Died September 23, 1930

Erastus Roland Simpson was born in Harpswell, Me., on November 29, 1870. He was the son of Erastus C. and Minerva (Trufant) Simpson, and a descendant of William and Agnes L. Simpson who settled at Simpson's Point, in Brunswick, Me., in 1728.

Mr. Simpson entered Brunswick High School in September, 1890. He completed a four-year course in June, 1893, and entered the University of Maine where he was enrolled in the Mechanical Engineering Department. In June, 1896, he was graduated from the University with the degree of Bachelor of Mechanical Engineering. In 1903, he submitted his thesis and received the degree of Mechanical Engineer.

---

[2] "Australian Ski Book," 1929.

[1] Memoir prepared from information on file at the Headquarters of the Society.

Directly following his graduation Mr. Simpson entered the employ of the Hancock Inspirator Company, of Boston, Mass., as a Draftsman on general machine design. After a period of six months with this Company, he resigned to become Assistant Engineer with the Metropolitan Park Commission, in Boston, for which he was engaged in drafting, field work, and as an Assistant Engineer on construction. About a year thereafter, he entered the employ of the Contractors' Plant Company, of Boston, as Head Draftsman on Mechanical Construction.

Subsequent to a year in the service of this Company he went to Georgetown, S. C., where he was employed as Engineer for the Atlantic Coast Lumber Company on mill construction. His work on this engagement was in designing, laying out, and superintending the construction of three mills, two power plants, machine shops, railroad shops, and docks.

Mr. Simpson was then employed by the late Freeman C. Coffin, M. Am. Soc. C. E., Consulting Hydraulic Engineer, of Boston, as an Assistant Engineer in general practice. In March, 1900, he accepted a position as Engineer for the Chapman Double Ball Shafting Bearing Company, of Boston. In this position his work was similar to that of his previous engagements, being mainly in designing and its resulting duties. On May 9, 1903, he went to Toronto, Ont., Canada, as General Manager of the Mechanical Department for this same Company.

In 1912, Mr. Simpson again engaged in the private practice of engineering as an Industrial Engineer, in Boston, with an office at 176 Federal Street, which he retained until his death on September 23, 1930.

Mr. Simpson was a member of the Boston Society of Civil Engineers, the Revere Lodge of Masons, in Boston, and Beta Theta Pi Fraternity. He was unmarried.

Mr. Simpson was elected an Associate Member of the American Society of Civil Engineers on November 4, 1903.

HAROLD DAVID STOLL, Assoc. M. Am. Soc. C. E.[1]

DIED JULY 9, 1930

Harold David Stoll, the son of Oscar P. Stoll and Alice (DePue) Stoll, was born on January 27, 1890, at Dalton, Pa. His early education was obtained in the public schools in the town of his birth and his technical training by private study and in the International Correspondence Schools of Scranton, Pa.

Mr. Stoll was first employed on the location of the Scranton and Binghamton Electric Railway, maintaining this connection until 1913, when he took a position with the Pennsylvania Department of Highways as Draftsman and Inspector on highway construction. In 1915, he associated himself with a local engineer in private practice in Scranton and vicinity. This work engaged his attention until 1916, when he went to the other end of the State to assist the Jones and Laughlin Steel Company in the development of town and property sites at Woodlawn, Beaver County.

---

[1] Memoir prepared by P. M. Tebbs, M. Am. Soc. C. E.

It was just about this time that the Pennsylvania Department of Highways was looking toward the development of the State's roadways on a rather extensive scale. In the northwestern corner of Pennsylvania an engineering district, comprising the Counties of Susquehanna, Wayne, Wyoming, Lackawanna, Pike, and Monroe, was created and, in 1918, Mr. Stoll was made Assistant District Engineer, with headquarters at Scranton, a position he filled so creditably that in 1921 he was made District Engineer. The importance of road construction was manifesting itself more and more each year and successive Legislatures were appropriating increasingly large sums of money for that purpose. It thus came about that Mr. Stoll had no small part in laying the foundation of one of the most important activities in which the Commonwealth has been engaged.

Every section of the State was clamoring to have its main arteries of travel put under a hard surface. It was a time when conscientious and impartial distribution of funds, and loyal service were necessary. Consideration was given to all demands, but early in the program came the development of the Lackawanna Trail, from Stroudsburg north to the Pocono Mountains *via* Scranton, to the New York line near Binghamton. This well-known route through some of the most beautiful scenery in the Eastern United States was one of the outstanding projects upon which Mr. Stoll was engaged and bears mute tribute to his tireless and indefatigable efforts. Commendation for this accomplishment is merited especially since during the latter part of his tenure in this office he was suffering from the first stages of a baffling malady which later necessitated the amputation of one leg. This did not suffice to check the progress of the disease, and in order that he might better contend with it, he severed his connection with the Pennsylvania Highway Department and went to Florida, where for a year, as a Professional Engineer of that State, he practiced in and around Daytona Beach.

In 1926, Mr. Stoll returned to the Pennsylvania Department of Highways as an Assistant Engineer in the Central Office at Harrisburg, Pa. His duties in the field had given him a peculiar insight into the problems confronted by the engineers and contractors performing the actual operations of road building; he was thus unusually well fitted to be liaison official to serve alike the interests of both the Department and the commercial agencies employed by it.

The physical indisposition which had assailed him in 1925 was held in abeyance until 1928. It re-asserted itself, however, and in order to effect what was thought might be a permanent cure, the other lower leg was amputated. Nothing daunted, and notwithstanding the entire loss of the means of natural locomotion, Mr. Stoll continued to perform his duties and spared no effort to promote his work as a servant of the public. His fortitude, cheerfulness, and strength of character became proverbial, his life serving as a restraining influence on those who indulge in fancied grievances and complaints.

The admiration of all who came in contact with him was spontaneous, a tribute to his loyalty, courage, and devotion to what he conceived to be his duty.

It was while he was enjoying a vacation at Lake Kewanee, in Lackawanna County, during the week of July 4, that he developed a severe case of pleurisy, the result of being caught in the rain while on a fishing trip. In his already weakened condition, he was unable to withstand this final illness, and death came very unexpectedly on the morning of July 9.

Mr. Stoll built for himself an enduring monument not only in his work as a public servant, but as a benefactor. Little is known of his kindnesses to the many who, less fortunate than he, received favors at his hands. He directed such efforts so quietly that only the recipients themselves were aware of what he did.

His widow, Helen R. (Stoltz) Stoll, of Jersey City, N. J., to whom he was married in 1913, and a daughter, Dorothy Rae Stoll, survive him.

Mr. Stoll was elected an Associate Member of the American Society of Civil Engineers on September 10, 1923.

FRANKLIN STEVENS STOREY, Assoc. M. Am. Soc. C. E.[1]

DIED OCTOBER 8, 1930

Franklin Stevens Storey, the son of the late Col. James Henry Storey, and Annie (Cheshire) Storey, was born at Brooklyn, N. Y., on December 15, 1881. He attended the public schools of Brooklyn and prepared there for Cornell University, from which he was graduated as a Civil Engineer in 1902.

On leaving college, Mr. Storey was appointed Leveler and Transitman with the Manhattan Elevated Railway Company, New York City; in October, 1903, he was employed as Assistant Engineer on the construction of the East River Tunnels, by the Rapid Transit Commission; and in September, 1904, he was engaged by the New York Dock Department as Assistant Engineer on the Chelsea Improvement, North River, and on the St. George, Staten Island, Ferry Terminal.

In June, 1906, Mr. Storey joined The Phoenix Construction Company as General Superintendent, laid out methods and details of construction, and had full charge of field work on the Company's numerous contracts in the vicinity of New York and along the Atlantic Coast. The work was generally of an interesting character and comprised piers, bulkheads, sea-walls, intakes, and discharge tubes for power plants, including the Ferry Terminal substructure, at Stapleton, Staten Island; the Subway Waiting-Room for the Erie Railroad Company, at Jersey City, N. J.; the Penhorn Creek Viaduct foundations for the main line of the Erie Railroad Company, over the Hackensack River; the Bronx Valley Sewer Outlet, at Yonkers, N. Y.; a bulkhead platform for the Ocean Steamship Company, Savannah, Ga., and Pier No. 5 for the West Shore Railroad Company, Weehawken, N. J.

On May 15, 1917, Mr. Storey was admitted to the first Officers' Training Camp, at Madison Barracks, New York, as a civilian candidate for a commission; he was assigned to the Engineer Company, 3d Provisional Training Regiment, moved with his organization to Camp American University, Washington, D. C., and served at that place until August 15, 1917, when he was appointed a First Lieutenant, Engineer Officers' Reserve Corps. He was assigned to active duty the same day and on August 27, 1917, reported for duty with the 303d Engineers.

---

[1] Memoir prepared by L. R. Gwyn, Esq., New York, N. Y.

He sailed overseas on September 14, 1917, for France where he was assigned to duty on October 4, 1917, at Le Valdahon as Post Engineer Officer. He served at that place until May 11, 1918, when he was transferred to Bazoilles-sur-Meuse, to assist in the construction of a hospital. He was transferred on July 5, 1918, to the Engineer Officers' Construction Camp in the 14th Divisional Area, Recey-sur-Ource, and on October 3, 1918, to the Engineer Officers' Construction Camp at the 2d Corps Aero School, Chatillon-sur-Seine.

On October 26, 1918, Lieutenant Storey was promoted to the rank of Captain, Engineers, and on November 4, 1918, he was transferred to Donjeux as Engineer Officer in charge of construction. On November 20, 1918, Captain Storey was transferred to Verdun and, later was on duty at Bordeaux. On March 7, 1919, he was assigned to the command of Company D, 539th Service Battalion, Engineers, relieved from duty therewith on May 16, 1919, and honorably discharged at Gievres, France, on June 18, 1919.

Returning to the United States, he immediately took up the position of Chief Engineer of The Phoenix Construction Company, which position he held until his death. Until his health failed, he had charge of work which included the first section of the Coney Island Boardwalk and the screen well and discharge tunnels for the New York Edison Company's Power House at 14th Street, East River. He died after a lingering illness, at Saranac Lake, N. Y., on October 8, 1930, and was buried in the family plot in Greenwood Cemetery, Brooklyn.

Captain Storey made friends for the Company on every "job" which he undertook and, what is rarer, kept them throughout his life. His mind was of an unusually fine caliber and he solved many a problem of a most difficult nature in both construction and design.

He is survived by his widow, his brother, Henry M. Storey, of San Francisco, Calif., and three sisters, Mrs. E. K. Waterbury and Mrs. L. R. Gwyn, both of Scarsdale, N. Y., and Mrs. P. S. Mahood, of Kingston, Ont., Canada.

Captain Storey was elected an Associate Member of the American Society of Civil Engineers on May 6, 1914. He was also a member of the Engineers Club of New York.

JOHN GERARD THEBAN, Assoc. M. Am. Soc. C. E.[1]

Died May 13, 1930

John Gerard Theban was born on May 26, 1869, in New York, N. Y., the son of Theodore Adrian and Alice M. (Casey) Theban. He was graduated from St. Francis Xavier College, New York, in June, 1890, with the degree of Bachelor of Arts.

---

[1] Memoir compiled from information on file at the Headquarters of the Society.

Mr. Theban's first position after his graduation was in the Engineering Department of the New York Central and Hudson River Railroad Company, where he began work in July, 1890. In June, 1892, he severed his connection with this Company and took a position with the Park Avenue Improvement Commission of New York City, which had control of all the work done in connection with the extensive changes made in the grades of the tracks of the New York and Harlem Railroad from Mott Haven Junction to 106th Street, along Park Avenue. In September, 1895, he re-entered the service of the New York Central and Hudson River Railroad Company as an Assistant Engineer for the New York and Harlem Railroad, the New York and Putnam Railroad, and its New York City Terminal System.

In June, 1896, he resigned his position with the Railroad Company to accept an appointment as Assistant Engineer of Construction in the Department of Street Improvements of the 23d and 24th Wards, New York City (now the Borough of The Bronx). In 1898, the first year of the City of Greater New York, Mr. Theban was transferred to the newly created Department of Bridges and was appointed Assistant Engineer in Charge of the Bridges over navigable waters in the Borough of The Bronx. From 1898 to 1912, in which latter year he became Engineer in Charge of the Harlem River and Bronx Division, when the two Divisions were consolidated, there were built in the Borough of The Bronx, under his charge, such structures as the City Island Bridge, the Westchester Avenue Retractile Draw Bridge, over the Bronx River, the fixed trunnion bascule bridge over Mott Haven Canal, at 135th Street, and the Scherzer rolling lift bridge with reinforced concrete arch approaches over Pelham Bay.

In January, 1913, Mr. Theban was made Engineer in Charge of the maintenance of the Queensboro Bridge, and in January, 1916, he became Engineer in Charge of the Williamsburg Bridge, both over the East River, New York City.

On April 30, 1916, he severed his connection with the City service to enter private practice. During the World War he was connected with the United States Navy Department as Supervising Engineer of Plant, Docks, and Structures, with headquarters at Akron, Ohio. At the time of his death, he was President of the Boretest Engineering Company, with offices in New York.

For years, Mr. Theban served as a Trustee and, later, became President of the McCadden-McQuirk Foundation, a trust fund established to assist deserving poor young men who wished to study for the Roman Catholic priesthood.

Mr. Theban was a man of refinement, a master of English, a skillful and accomplished engineer, and an efficient executive. He was also an accomplished pianist and a great lover of classical music.

In June, 1904, he was married to Sarah Lawrence Gunning, of South Orange, N. J. He is survived by his widow, two daughters, Alice and Evelyn, one son, John, and a brother, Theodore A. Theban.

Mr. Theban was elected an Associate Member of the American Society of Civil Engineers on March 3, 1897.

WALTER CHECKLEY TIFFANY, Assoc. M. Am. Soc. C. E.[1]

DIED DECEMBER 9, 1929

Walter Checkley Tiffany, Jr., was born at Minneapolis, Minn., on January 23, 1891. His father is Walter C. Tiffany, now of Seattle, Wash., and his mother was Lila R. (Robinson) Tiffany. His early years were spent in the city of his birth, where his father practiced law. His primary education was by private tutor and in various private and public high schools. Although the tradition of his family inclined toward education at Harvard University, of which his father is a graduate, his own restless and impatient temperament early led him into the practical field of construction work. Later, he sensed the advantage to him of technical training, and he tied himself down for a period of study at the School of Mines at Butte, Mont. All through his active and adventurous life Mr. Tiffany was an assiduous student in his spare time. He had an insatiable appetite for facts, figures, and the experiences of others as set forth in technical articles and particularly in descriptions of construction work.

In his early professional life which began at the age of seventeen, Mr. Tiffany went through the usual progression of the "cub", as Chainman, Rodman, and Transitman on railroad location and construction in the Western States, filling in between jobs as Computer and Draftsman. His first engagement was as Chainman on the Los Angeles Aqueduct, in 1909. The next year he was with the Northern Pacific Railroad Company, in North Dakota; the following year on the Duluth-Frederick Division of the Milwaukee, St. Paul, and Sault Ste. Marie Railway, and at Wilton, Wis., on the Ontario Northern Railway. In 1912 and 1913 he was engaged as Instrumentman with the Southern Traction Company from Dallas to Waco, Tex., and during the following two years he was with the Pacific Telegraph and Telephone Company, at Seattle, and with the Standard Oil Company at Waco. At the outbreak of the World War he was employed as Topographer with the Stone and Webster Engineering Corporation in the Seattle District

As a Private and Sergeant in the Washington National Guard, Mr. Tiffany was prepared for the call to the Service. He attended the Third Officers' Training School at Camp Lewis, Washington, after which he was placed in the Field Artillery Replacement Depot at Camp Jackson, South Carolina. In May, 1918, he sailed for France where he attended the Saumur Artillery School. In September of that year he was assigned to the 330th Field Artillery and served as Second Lieutenant throughout the series of engagements in the Marne Region. He returned to the United States in April, 1919, and was honorably discharged at Camp Grant, Illinois.

The next eleven years was a period of almost excessive activity for Mr. Tiffany, with progressive increase of responsibility on a large number of construction jobs, including location work on the Minarets and Western Railway, in California; surveys and construction of the Caribou and Big Creek

---

[1] Memoir prepared by George Schobinger, M. Am. Soc. C. E.

Developments, in California; irrigation work, in Mexico; power plant and mining smelter development, in California and Arizona; and, later, in New York City, where he was Superintendent with Dwight P. Robinson and Company, Incorporated, on a large steam boiler plant. In 1927, he was assigned by that Company to work in Rio de Janeiro, Brazil, where he became its General Superintendent and Representative, in connection with the construction of a number of buildings, and of a large new shop development.

In all his assignments, and particularly in the later ones where the opportunities were becoming greater, Mr. Tiffany showed single-mindedness of purpose, capacity for work, and admirable ability to organize and to apply the results of his early experience, which appeared to assure a brilliant future. In his last year, however, perhaps as a result of overwork, exposure, and hardship in his earlier years and during the war period, a serious disability of his heart and lungs began to appear, which, at times, would strike him down at moments of intense activity. He refused to believe that he could have any physical limitations, and in spite of warnings from within and from his medical advisers, drove himself perhaps harder than ever. His only interest and his only desire in life was to finish successfully this last and largest job which he had organized and put in full working order. In November, 1929, he received an unmistakable warning of physical inability and consented to go to bed for ten days, but even then kept his bed stacked with reports and documents, and had the telephone within reach. The forced inactivity was too much; he snapped his fingers at the doctor and announced that he was going to "run" his job in spite of doctors. A week later he was fatally stricken a few minutes after returning home from work, and two days later, on December 9, 1929, died at the Strangers' Hospital, in Rio de Janeiro.

In 1923, Mr. Tiffany was married to Margaret Curtis, who shared with him the life of an engineer, "from town to camp and from job to job." Friends told him that in the intensity of his work, which occupied every daylight hour, and, in addition, many hours of the night, he was neglecting his family, but he knew that he was understood by his wife, and that he had her unwavering support and help in doing what he considered necessary to his job. He discussed many of his problems at home and benefited by the balance of judgment which comes from the application of a new point of view and a fresh mind.

He had a keen mind and a quick temper under perfect control; in many years of association none of his men ever heard his voice raised in anger, although there never was any question about the firmness of his discipline or his requirements that his orders be properly carried out. A "horse" for work, careful in detail, thoroughly familiar with costs, ambitious, a good disciplinarian, and able to enlist the co-operation and affection of all the men in his organization, he was an ideal construction engineer.

Walter Tiffany was an exceptional man. One of his former associates, answering inquiries as to his record, closed his letter with the following paragraph:

"From out of my profound admiration for Walter as an Engineer and my genuine affection for him as a friend there is little that I can add, except the words of Antony about his friend, that 'This was a man'."

The same associate wrote: "It has seldom been my good fortune to be associated with so capable, so earnest, and so energetic an Engineer."

He is survived by his widow and two small sons, Walter Checkley, Jr., and Francis Tiffany.

Mr. Tiffany was elected an Associate Member of the American Society of Civil Engineers on October 12, 1925.

WILLIAM FRANKLIN WHITAKER, Assoc. M. Am. Soc. C. E.[1]

DIED NOVEMBER 20, 1930

William Franklin Whitaker, the son of David S. and Annie (Warren) Whitaker, was born on January 20, 1880, at Lebanon, Ind. He was educated in the schools of Lebanon and in 1899 entered Leland Stanford, Jr., University, California, from which he was graduated in June, 1903, with the degree of Bachelor of Arts in Civil Engineering.

Mr. Whitaker's first engineering work began when he was only nineteen years of age, and prior to his entering college, at which time he was employed on engineering parties for the Kern County Land Company, Bakersfield, Calif., and also with the City Engineer of Bakersfield. In 1902, during his college vacation, he was engaged also in engineering work, as Stream Gauger for the Buena Vista Lake Reservoir Association, measuring the flow of Kern River, and, under the supervision of the late Edwin Duryea, M. Am. Soc. C. E., on underground water measurements and pumping tests in the Santa Clara Valley, California.

After graduating from Stanford University, Mr. Whitaker was first employed, in 1903, as Draftsman in connection with reservoir surveys and, later, in charge of parties engaged on the measurement of stream flow in the Coyote River and other streams tributary to the Santa Clara Valley. In December, 1904, he accepted a position as Assistant Engineer with the Kern County Land Company of Bakersfield, of which Mr. Walter James was Chief Engineer. His work in this capacity involved surveys and a study of the irrigation system of Kern Delta. For two months in 1905, he also assisted in a study of the water-works system of the City of Bakersfield and the preparation of a full report on existing conditions, with recommendations for enlargement and betterment of the system for fire protection, etc.

From November, 1905, until 1917, Mr. Whitaker was Superintendent of Canals, and, from 1917 to the date of his death, Chief Engineer of the Kern County Land Company, having full charge of the maintenance and operation of more than 1 000 miles of canals and laterals serving about 200 000 acres of land in Kern County, California. In connection with this engagement, he designed and constructed numerous irrigation works and structures of magnitude, administered the irrigation system, and also acted in a consulting capacity on various other irrigation projects in California, Arizona, and New Mexico.

---

[1] Memoir prepared by a Committee of the San Francisco Section consisting of H. L. Haehl, M. Am. Soc. C. E., and Hugh S. Allen and A. L. Trowbridge, Assoc. Members, Am. Soc. C. E.

Mr. Whitaker was possessed of an especially pleasant personality, unusual force of character, and marked engineering ability. He will be particularly remembered among the young men of the profession who came in contact with him for his unselfish interest in their personal welfare and the untiring assistance and kindly advice he freely afforded them in their work. He was consistently interested and active in the affairs of the Society. He was also intensely active and influential in his community in civic, business, financial, and educational affairs. He was devoted to his family and his many friends, and leaves behind him an inspiring record of good works and charitable acts. He died of a cerebral hemorrhage on November 20, 1930, when apparently in excellent health and in the midst of an active and useful life.

Mr. Whitaker was married on August 10, 1907, to Gertrude Scribner, of Bakersfield, who, with four children—Elizabeth, Anne, William, and Ruth—survives him. He is also survived by his parents, a brother, Ralph W. Whitaker, Assoc. M. Am. Soc. C. E., of San Diego, Calif., and a sister, Mrs. W. E. Duthie, of Indianapolis, Ind.

Mr. Whitaker was elected a Junior of the American Society of Civil Engineers on April 4, 1905, and an Associate Member on November 6, 1910.

ROBERT ELGENE YOLTON, Assoc. M. Am. Soc. C. E.[1]

DIED SEPTEMBER 10, 1930

Robert Elgene Yolton was born in Fayette County, Illinois, on December 25, 1879. He lived on a farm until he was twenty years old, at which time he entered Austin College, at Effingham, Ill., where he spent three years. He then went to the University of Illinois as a student in Civil Engineering, from which he was graduated in 1905.

After his graduation, Mr. Yolton was engaged with the Buda Company in Chicago, Ill., where he spent six years. He then worked for a short time for the Illinois Central Railroad Company, also in Chicago.

In the fall of 1916 he was called to Birmingham, Ala., as Chief Engineer of the Kilby Frog and Switch Company. Mr. Yolton spent ten years with this firm, during which time he did much to improve the plant; in fact, under his guidance it was increased to about twice its original size. In the summer of 1925 the Company was consolidated with the Weir Frog Company of Cincinnati, Ohio, and, in January, 1926, Mr. Yolton went to Cincinnati to become Chief Engineer of the Weir-Kilby Corporation, with which he remained until his death.

He was greatly interested in flowers and spent much time in his garden, which was always greatly admired by his friends. He was also active in church affairs and was an Elder in Presbyterian Churches in Chicago, Birmingham, and, at the time of his death, in the Norwood (Ohio) Presbyterian Church. He was a man who was greatly loved and admired by his friends. Mr. Kilby, former President of the Kilby Frog and Switch Company, said of him:

---

[1] Memoir prepared from information on file at the Headquarters of the Society.

"Mr. Yolton was a man of sterling character and of the greatest integrity. He was loved and respected equally by his employers as well as by his subordinates to whom he was always counsellor and friend."

Mr. Yolton was a member of Tau Beta Pi Fraternity and the National Geographic Society.

He was married in Melvin, Ill., on August 30, 1904, to Ella Maude Holmes, who, with two sons, Robert H. and John W., and a daughter, Jean Mildred, survives him.

Mr. Yolton was elected an Associate Member of the American Society of Civil Engineers on April 17, 1917.

JAMES RUSSELL VAUGHAN, Jun. Am. Soc. C. E.[1]

DIED DECEMBER 16, 1930

James Russell Vaughan, the only child of Thomas M. Vaughan and Mae (McReynolds) Vaughan, was born in Indianapolis, Ind., on March 14, 1904. He was graduated from the High School at Lisbon, Ohio, with the Class of 1921.

From early boyhood, bridges and the building of bridges interested him greatly. By the time he had finished his high school studies, this interest had grown so strong that he decided to take up Engineering and devote himself to the design and construction of bridges. With this in mind he entered the Case School of Applied Science in the fall of 1921.

Lack of funds prevented Mr. Vaughan from completing the course in the customary four years. However, with characteristic energy and persistence, he worked his way through and received the degree of Bachelor of Science in Civil Engineering with the Class of 1927. While at College, he was much interested in athletics, having been a star halfback on the Case football team and a consistent performer in the dashes and hurdles. He was a member of the Kappa Sigma Fraternity.

From October, 1923, to September, 1924, Mr. Vaughan was in the employ of The Ohio Power Company, at Canton, Ohio, engaged in making surveys, checking property ownership, and mapping the right of way of the Company.

In August, 1927, he entered the drafting-room of The Mt. Vernon Bridge Company, at Mt. Vernon, Ohio, as a detailer of steel work. He was blessed with an unusually active mind and a strong and healthy body which enabled him to endure long hours of work and study. He not only did his office work faithfully and well, but spent most of his evenings and holidays studying the theory of structures and reading all available books on the design of bridges. Many times he worked far into the night checking the design of some bridge which he was detailing during working hours or working out a design for some bridge about which he had read.

Mr. Vaughan's ability attracted the attention of the older engineers of the Company and from time to time when extra help was needed in the Designing Department, he was chosen. The two most important contracts on which he assisted with the design were a 345-ft. swing span bridge for the Baltimore and Ohio Railroad Company over Curtis Creek near Baltimore, Md., and the equipment to be used in the erection of the 750-ft. three-hinged arch for the Westchester County Park Commission over Croton Lake near New York, N. Y.

---

[1] Memoir prepared by James D. K. Lyman, M. Am. Soc. C. E.

The erection of this arch was of such a nature that the Mt. Vernon Bridge Company required the services of an engineer on the construction during erection. Mr. Vaughan being thoroughly familiar with the erection procedure was chosen for this work. During his work in the field, he won the respect and admiration of both the workmen and the other engineers with whom he came in contact.

On December 11, 1930, while Mr. Vaughan was checking the elevation of the top of a bent of falsework, a wrench slipped from the hands of a workman at the next panel point ahead and, in falling, struck the sloping bottom chord of the arch and glanced sidewise striking Mr. Vaughan on the head. The blow stunned him to such an extent that he fell to the ground, causing internal injuries from which he died on December 16, 1930.

His death is a distinct loss, not only to his friends and associates, but also to the Engineering Profession, for he gave promise of more than ordinary ability, and it is greatly to be regretted that a young life of so many possibilities should end so soon.

He is survived by his widow, Elizabeth Dieringer Vaughan, to whom he was married on Christmas Day, 1927, and his father and mother.

Mr. Vaughan was elected a Junior of the American Society of Civil Engineers on November 11, 1929.

STEDMAN BENT, Affiliate, Am. Soc. C. E.[1]

DIED NOVEMBER 26, 1929

Stedman Bent was born in what is now the City of Chester, Pa., on June 10, 1876, the son of Luther Stedman and Mary Stearns (Felton) Bent.

After attending school in Philadelphia, Mr. Bent went to work, in 1896, for the Pennsylvania Steel Company at its Steelton Works. Two years later, in 1898, he was appointed Purchasing Agent of that Company and its various subsidiary companies, with offices in Philadelphia. He held this position until about 1914, when he became connected with the Raymond Concrete Pile Company and was placed in charge of its office for the Philadelphia District. He remained with this Company until his death on November 26, 1929, at his home in Overbrook, Pa.

Mr. Bent was a member of the Racquet Club, the Philadelphia Country Club, the Pennsylvania Athletic Club, the Seaview Country Club, the Merion Cricket Club, and the Lotos Club of New York. He was also a member of the Sons of the Revolution and the Loyal Legion.

He was a lover of animals. He took an active interest in raising fine cattle and was extremely fond of dogs, always having several of them around his home. He was also greatly interested in farming and found great joy in his garden, to which he devoted much time.

---

[1] Memoir prepared by Edgar C. Felton, Esq., Philadelphia, Pa., John V. W. Reynders, Esq., New York, N. Y., and Thomas Earle, M. Am. Soc. C. E.

He was married on August 4, 1919, to Nanna (McElwell) Arnold, who survives him.

Mr. Bent was elected an Affiliate of the American Society of Civil Engineers on December 2, 1902.

CLARENCE MARVIN FOSTER, Affiliate, Am. Soc. C. E.[1]

Died February 18, 1930

Clarence Marvin Foster, the son of Maj.-Gen. Robert S. Foster and Margaret (Foust) Foster, was born in Indianapolis, Ind., on April 6, 1866. He completed his common and high school education in 1883.

From 1883 to 1887, Mr. Foster was associated with the firm of R. S. Foster Company, of Indianapolis, dealers in cement and building materials. In 1887, he started in business for himself as a General Contractor on concrete and masonry work. He continued in this business until 1892, when he disposed of it to become associated with the firm of Buntin, Shryer and McGannon, of Indianapolis, Duluth, Minn., and Kansas City, Mo., with which he remained until 1895. During his services with these various organizations, Mr. Foster had qualified himself as an expert on matters pertaining to cement and work in which cement was involved. Recognizing his value as an expert in matters pertaining to this type of work, he was engaged by the Utica Cement Association, manufacturers of hydraulic cement, as a Traveling Inspector. This work caused him to travel throughout the West.

In 1899, Mr. Foster left the Utica Cement Association to become Sales Manager for the Meacham and Wright Company, of Chicago, Ill. This Company was for many years one of the largest (if not the largest) jobbers of Portland cement in the United States. He continued in this capacity until the early part of 1914, when the Meacham and Wright Company discontinued handling Portland cement. He then became Manager of the Edison Portland Cement Company, at Philadelphia, Pa.

In April, 1915, Mr. Foster and J. Edward Cassidy, M. Am. Soc. C. E., organized the Foster Construction Company with headquarters in Newark, N. J., and engaged in the design and construction of reinforced concrete bridges. Mr. Foster, as President, was specifically charged with office administration and the purchase of materials and supplies for the operations of the Company. In the spring of 1917, when the United States entered the World War, the Foster Construction Company had a well-established business, and, at the time war was declared, had a number of bridges under construction. Mr. Cassidy, who was in charge of all the technical and construction work of the Company, was a Captain (later, Colonel) in the Engineer Reserve Corps and went on active duty the day following the finishing and acceptance of the last of the bridges under construction.

---

[1] Memoir prepared by J. Edward Cassidy, M. Am. Soc. C. E.

Although he was 51 years of age when war was declared, it was particularly fitting that Mr. Foster should take an active part in the service of the country, as his father, Maj.-Gen. Foster had had a very distinguished record in the Civil War serving in various capacities up to and including the command of a Division. His service further included the position of Chief of Staff of the 10th Army Corps, and afterward that of Major-General commanding the First Division, 24th Army Corps, Army of the James; this Division performed a 36-mile march in 24 hours to close the last gap around the Army of Northern Virginia at Appomattox, April 8 and 9, 1865.

Being especially fitted for work in connection with materials and supplies used in construction, Mr. Foster received an appointment in the Construction Division of the Army, was commissioned as a Captain, and, later, promoted to the rank of Major. He continued in the capacity of Purchasing and Contracting Officer, Construction Division, until October, 1919, when he left the service to become District Manager of the Barber Asphalt Company, with headquarters at Washington, D. C.; he remained with that Company until his death. During the period, 1919 to 1923, while his headquarters were in Washington, Major Foster compiled a great many valuable data on municipal work for which he received commendation not only from his own Company, but from a number of foreign countries.

In October, 1923, Major Foster was promoted to be District Manager of the Barber Asphalt Company on the Pacific Coast with headquarters at San Francisco, Calif., his territory covering a large group of Western States, the Hawaiian and Philippine Islands.

His death on February 18, 1930 occurred from a stroke of apoplexy. He had not been feeling well for days and on February 13, 1930, he came home early in the afternoon and shortly afterward suffered the stroke from which he never regained consciousness. He died five days later. Funeral services were held at his home in San Francisco, after which all that was mortal was taken to his old home in Indianapolis, and interred in Crown Hill Cemetery.

He was a member of the Military Order of the Loyal Legion, Indiana Engineering Society, and the U. S. Army Quartermaster Association.

Major Foster was a man of exemplary habits, and with his genial, pleasant disposition and courteous manner, had the faculty of making a host of friends wherever he went. His passing is a personal loss not only to his friends in many parts of the country, but to his numerous business associates with whom he was always a "hale fellow well met."

He was married on December 24, 1895, to Mary Margaret Roberts, of Indianapolis, who survives him, together with a daughter, Mrs. Maryanna Childs, of Detroit, Mich., and a son, Robert S. Foster, of San Francisco.

Major Foster was elected an Affiliate of the American Society of Civil Engineers on January 31, 1899.

11

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

INDEX

VOLUME 95

1931

SUBJECT INDEX, PAGE 1668
AUTHOR INDEX, PAGE 1682

Titles of papers are in quotation marks when given with the author's name.

VOLUME 95

SUBJECT INDEX

ADDRESSES.

Presidential Address at the Annual Convention at Tacoma, Washington, July 8, 1931. Francis Lee Stuart. 1294.

AIRPORTS.

"Factors Governing the Location of —." Donald M. Baker. 269. Discussion: Stephen Child, Morris Knowles, George D. Burr, N. H. Angell, J. Ogden Marsh, Archer R. Norcross, John Nolen, S. N. Grimm, R. U. St. John, John F. Partridge, William J. Fox, Perry A. Fellows, W. W. Crosby, E. K. Smith, and Clyde M. Wood. 279.

ARCHES.

"Laminated Arch Dams with Forked Abutments." Fred A. Noetzli. (With Discussion.) 533.

"Plastic Flow in Concrete —." Lorenz G. Straub. 613. Discussion: B. F. Jakobsen, C. A. P. Turner, Edward Godfrey, J. H. Griffith, Charles W. Comstock, A. A. Eremin, J. Melan, Fredrik Vogt and Herbert J. Gilkey, E. Probst, Paul Bauman, G. C. Staehle, and A. Floris. 679.

BARS.

"The Training Wall Across the Liao Bar in Manchuria." P. N. Fawcett. (With Discussion.) 969.

BIBLIOGRAPHY.

— of current-meter characteristics by American investigators. 791.

— of ice in hydro-electric plants. 1145.

— of Niagara Falls power plants. 26.

BRIDGE PIERS.

"Construction of the James River Bridge Project." R. C. Wilson and Herbert B. Pope. (With Discussion.) 244.

BRIDGES.

— on the Chesapeake and Delaware Canal. 740 *et seq.*

"Construction of the James River Bridge Project." R. C. Wilson and Herbert B. Pope. 244. Discussion: Wilson T. Ballard, and William Willoughby. 262.

"Impact in Highway —. Final Report of the Special Committee." (With Discussion.) 1089.

"Plastic Flow in Concrete Arches." Lorenz G. Straub. (With Discussion.) 613.

BUILDINGS.

"Florida Hurricane. Final Report of the Committee of the Structural Division." (With Discussion.) 1118.

CANALS.

"The Chesapeake and Delaware Canal." Earl I. Brown. 716. Discussion: F. T. Chambers. 764.

CITY PLANNING.

"— as Related to the Smaller Cities." E. A. Wood. 1075. Discussion: John L. Starkie, O. H. Koch, and Stephen Child. 1080.

CIVIL ENGINEERS.

"The Engineer's Growing Civic Responsibilities": Address at the Annual Convention at Tacoma, Washington, July 8, 1931. Francis Lee Stuart. 1294.

COLUMNS.

"Steel Column Research. Second Progress Report of the Special Committee." (With Discussion.) 1152.

CONCRETE.

"Baldwin Filtration Plant, Cleveland, Ohio." J. W. Ellms, G. W. Hamlin, A. G. Levy, and J. E. A. Linders. (With Discussion.) 459.
— lining, Moffat Tunnel. 353.
Construction methods and plant layout at Coolidge Dam, in Arizona. 159.
Cracking of — in dams. 145.
"Plastic Flow in — Arches." Lorenz G. Straub. (With Discussion.) 613.

CONTRACTORS PLANT.

Construction methods and plant layout at Coolidge Dam, in Arizona. 159.

COSTS OF WORK.

Analysis of costs of the Baldwin Filtration Plant, Cleveland, Ohio. 512 *et seq.*
Comparative costs of different types of highways in and adjacent to New York City. 1050.
Cost data, Moffat Tunnel. 369.
"Factors Governing the Location of Airports." Donald M. Baker. (With Discussion.) 269.

CURRENT METERS.

See **METERS.**

DAMS.

Construction methods and plant layout at Coolidge Dam, in Arizona. 159.
"High —": A Symposium. A. J. Wiley, D. C. Henny, F. L. Ransome, and J. G. Tripp. 129. Discussion: F. G. Switzer, Fred A. Noetzli, Edward Godfrey, B. F. Jakobsen, Allen Hazen, H. de B. Parsons, P. Wilhelm Werner, C. A. P. Turner, F. W. Hanna, H. F. Dunham, Lars R. Jorgensen, William P. Creager, L. F. Harza, A. Floris, and Joseph Wright. 169.

Ice thrust on up-stream faces of —. 1143.

"Laminated Arch — with Forked Abutments." Fred A. Noetzli. 533. Discussion: Lars R. Jorgensen, William Cain, B. F. Jakobsen, P. Wilhelm Werner, H. B. Muckleston, Fredrik Vogt, Paul Bauman, F. W. Hanna, and A. A. Eremin. 565.

"The Shannon Power Development in the Irish Free State." A. R. C. Markl. (With Discussion.) 597.

DIKES.

— on Chesapeake and Delaware Canal. 738.

DRAINAGE.

"A National Reclamation Policy. Report of the Committee of the Irrigation Division." With Explanatory Statements: J. B. Lippincott, and Elwood Mead. (With Discussion.) 1303.

DREDGING.

"The Chesapeake and Delaware Canal." Earl I. Brown. (With Discussion.) 716.

DRILLING.

— operations, Moffat Tunnel. 344.

EARTHQUAKES.

— in relation to dams. 144, 156.

EARTHWORK.

"The Shannon Power Development in the Irish Free State." A. R. C. Markl. (With Discussion.) 597.

ECONOMICS.

"A National Reclamation Policy. Report of the Committee of the Irrigation Division." With Explanatory Statements: J. B. Lippincott, and Elwood Mead. (With Discussion.) 1303.

Railways *versus* waterways: An economic comparison. 877.

"The Engineer's Growing Civic Responsibilities": Address at the Annual Convention at Tacoma, Washington, July 8, 1931. Francis Lee Stuart. 1294.

ELECTRIC POWER.

"Niagara Power." Norman R. Gibson. (With Discussion.) 1.

"The Shannon Power Development in the Irish Free State." A. R. C. Markl. (With Discussion.) 597.

EMBANKMENT.

"The Shannon Power Development in the Irish Free State." A. R. C. Markl. (With Discussion.) 597.

ENGINEERS.
"The Engineer's Growing Civic Responsibilities": Address at the Annual Convention at Tacoma, Washington, July 8, 1931. Francis Lee Stuart. 1294.

EROSION.
"Silting and Life of Southwestern Reservoirs." R. G. Hemphill. (With Discussion.) 1060.

EVAPORATION.
"— as a Function of Insolation." Burt Richardson. 996. Discussion: H. H. Kimball, C. G. Abbot, D. P. Krynine, R. C. Wells, and N. W. Cummings. 1012.
"Rainfall Characteristics and Their Relation to Soils and Run-Off." C. S. Jarvis. (With Discussion.) 379.

EXCAVATION.
Construction methods and plant layout at Coolidge Dam, in Arizona. 159.
— at Shannon Power Development, Irish Free State. 603.

FILTERS.
See **FILTRATION.**

FILTRATION.
"Baldwin — Plant, Cleveland, Ohio." J. W. Ellms, G. W. Hamlin, A. G. Levy, and J. E. A. Linders. 459. Discussion: S. M. Van Loan, Harry N. Jenks, James H. Herron, and E. Sherman Chase. 519.

FINANCE.
"A National Reclamation Policy. Report of the Committee of the Irrigation Division." With Explanatory Statements: J. B. Lippincott and Elwood Mead. (With Discussion.) 1303.

FLEXURE.
See **STRESS.**

FOUNDATIONS.
"Experimental Research on Vibration Dampers and Insulators." Stephen E. Slocum. (With Discussion.) 85.

FRAZIL.
See **ICE.**

FREIGHT.
Railways *versus* waterways: An economic comparison. 877.

FRICTION.
Air resistance in Moffat Tunnel. 359.

GAUGES.

Recording rain gauge at Chestnut Hill Reservoir, Boston, Mass. 952.

GEOLOGY.

Geological conditions encountered in Moffat Tunnel. 342.

High dams: The viewpoint of the geologist. 149.

HARBORS.

"The Training Wall Across the Liao Bar in Manchuria." P. N. Fawcett. (With Discussion.) 969.

HIGHWAYS.

"— as Elements of Transportation." Fred Lavis. 1020. Discussion: W. W. Crosby, Walter Loring Webb, Jay Downer, S. Johannesson, D. P. Krynine, Harold M. Lewis, Donald M. Baker, Edwin F. Wendt, and H. W. Hudson. 1041.

Through trunk highway through Jersey City and Newark. 1020.

HURRICANES.

"Florida Hurricane. Final Report of the Committee of the Structural Division." (With Discussion.) 1118.

HYDRO-ELECTRIC POWER PLANTS.

See **POWER PLANTS.**

ICE.

Bibliography of — in hydro-electric plants. 1145.

"— as Affecting Power Plants. Final Report of Committee of Power Division." (With Discussion.) 1134.

IMPACT.

"— in Highway Bridges. Final Report of the Special Committee." (With Discussion.) 1089.

INSOLATION.

"Evaporation as a Function of —." Burt Richardson. (With Discussion.) 996.

INSULATION.

"Experimental Research on Vibration Dampers and Insulators." Stephen E. Slocum. (With Discussion.) 85.

IRRIGATION.

"A National Reclamation Policy. Report of the Committee of the Irrigation Division." With Explanatory Statements: J. B. Lippincott, and Elwood Mead. (With Discussion.) 1303.

JETTIES.

— at outlet of Chesapeake and Delaware Canal. 737.
"The Training Wall Across the Liao Bar in Manchuria." P. N. Fawcett. 969. Discussion: R. D. Goodrich, Herbert Chatley, H. C. Ripley, and B. Okazaki. 984.

LAKES.

Effect of power development on Lake Erie and the Niagara River. 16.
"Regulation of Levels, Flow, and Navigation, Niagara River; Summary and Conclusions of Various Studies." George B. Pillsbury. (With Discussion.) 51.

LAND RECLAMATION.

"A National Reclamation Policy. Report of the Committee of the Irrigation Division." With Explanatory Statements: J. B. Lippincott, and Elwood Mead. (With Discussion.) 1303.

LAND SLIDES.

— on Chesapeake and Delaware Canal. 733.

LIFT BRIDGES.

Bridges on the Chesapeake and Delaware Canal. 740, *et seq.*

MEMOIRS OF DECEASED MEMBERS.

Allen, Kenneth. 1437.
Allison, William Franklin. 1440.
Anderson, Justin Kenderdine. 1442.
Ash, Henry Clarke. 1443.
Ashmead, Frank Milligan. 1445.
Axtell, Frank Foy. 1446.
Barnes, Mortimer Grant. 1449.
Bent, Stedman. 1663.
Berquist, Axel Samuel Frederick. 1451.
Black, Alexander Leslie. 1452.
Bland, John Carlisle. 1453.
Bowers, George. 1456.
Bratager, Stoerk Johan. 1457.
Brendlinger, Peter Franklin. 1459.
Brown, Robert. 1462.
Burr, Henry Amsden. 1463.
Byers, Maxwell Cunningham. 1465.
Cain, William. 1467.
Campen, George Linden. 1473.
Carson, William Waller. 1475.
Cooley, Maurice Wurtz. 1477.
Davidson, George. 1428.
Dodge, Grenville Mellen. 1432.
Duryea, Edwin, Jr. 1478.
Edwards, Raymond Arden. 1634.

Fehr, Harrison Robert, Sr. 1480.
Felton, Samuel Morse. 1481.
Foster, Clarence Marvin. 1664.
Freeman, Roger Morse. 1485.
Fulton, William James. 1636.
Gates, Horace Delphos. 1488.
Gerdine, Thomas Golding. 1490.
Gilmore, Thomas Nixon. 1492.
Goodwin, Arthur Bates. 1637.
Hall, Benjamin Mortimer. 1493.
Hart, Charles Ellett. 1639.
Harvey, Alfred Ernest. 1496.
Hawgood, Harry. 1497.
Hazen, Allen. 1500.
Hemphill, Robert Grier. 1639.
Herschel, Clemens. 1419.
Hill, Albert Banks. 1503.
Hill, Edward Baxter. 1641.
Hill, Edwin Allston. 1642.
Hill, John Wilmuth. 1506.
Hogeland, Albert Harrison. 1509.
Holgate, Henry. 1511.
Hoxie, Richard Leveridge. 1513.
Hoyt, William Hausmer. 1515.
Huckaby, Marion Columbus. 1517.
Hughes, Hector James. 1518.
Jennings, John Edward. 1520.
Junkersfeld, Peter. 1522.
Kinsley, Thomas Pearson. 1524.
Kreutzer, George Charles. 1526.
Kwong, King Yang. 1528.
Long, Eugene McLean. 1531.
Lotter, Henry Howell. 1533.
Ludlow, Jacob Lott. 1536.
Lundie, John. 1538.
MacKay, Henry Martyn. 1541.
Maclay, William Walter. 1543.
McCarthy, George Arnold. 1545.
McClintock, William Edward. 1546.
McCoy, Laurence Francis. 1549.
McDonald, John Alexander. 1550.
McGrew, Anson Burlingame. 1552.
Mandigo, Clark Rogers. 1553.
Manson, Marsden. 1554.
Martin, Charles Cyril. 1556.
Minor, Sheldon Elton. 1558.
Mitchell, Henry. 1560.
Moncrieff, John Mitchell. 1561.
Moulton, Guy. 1563.
Münster, Andreas Wendelbo. 1565.
Nickerson, Ralph Waldo. 1566.

Norman, George H. 1568.
Norton, George Harvey. 1569.
O'Connor, Cornelius Joseph. 1644.
Parks, Charles Wellman. 1571.
Polhemus, James Suydam. 1575.
Pope, James Bond. 1576.
Porter, George Frederick. 1577.
Porter, Waldo Justin Bickle. 1645.
Pratt, William Abbott. 1579.
Pugh, Marshall Rogers.. 1581.
Raitt, John William. 1584.
Reed, William Boardman. 1585.
Rhodes, Claude Irvin. 1586.
Riffle, Franklin. 1587.
Rix, Edward Austin. 1590.
Rodd, Thomas. 1592.
Rode, Hans Henrik. 1594.
Rogers, Joseph Warren. 1646.
Rostock, John Henry. 1595.
Rousseau, Harry Harwood. 1597.
Russell, Richard Lord. 1601.
Samuel, George Frederick. 1603.
Scripko, Nicholas Alexander. 1649.
Seaman, William Laurie. 1650.
Shaffer, Ivan Oscar. 1604.
Sharts, Stanley Rush. 1605.
Simpson, Erastus Roland. 1652.
Smith, Bronson Hasbrouck. 1606.
Stoll, Harold David. 1653.
Storey, Franklin Stevens. 1655.
Stubbs, Linton Waddell. 1608.
Taylor, Harry. 1609.
Theban, John Gerard. 1656.
Tiffany, Walter Checkley. 1658.
Tribus, Louis Lincoln. 1612.
Turner, Edmund Kimball. 1614.
Van Keuren, Charles Augustus. 1615.
Van Sant, Robert Lawrence. 1616.
Van Zile, Harry Lee. 1617.
Vaughan, James Russell. 1662.
Vincent, Edward Franklin. 1619.
Webster, Albert Lowry. 1621.
Webster, George Smedley. 1423.
Whitaker, William Franklin. 1660.
Whitfield, James Edward. 1622.
Wight, Frank Clinton. 1623.
Williams, Frank Martin. 1626.
Williams, William Fish. 1628.
Witt, Carlton Carpenter. 1630.
Yeatman, Morgan Edward. 1632.
Yolton, Robert Elgene. 1661.
Young, Samuel McCain. 1633.

METEOROLOGY.

"Rainfall Characteristics and Their Relation to Soils and Run-Off." C. S. Jarvis. (With Discussion.) 379.

METERS.

Bibliography of current-meter characteristics by American investigators. 791.

"Effect of Turbulence on the Registration of Current —." David L. Yarnell and Floyd A. Nagler. 766. Discussion: Charles S. Bennett, Carl Rohwer, Brehon B. Somervell, N. C. Grover, Burke L. Bigwood, G. H. Nettleton, C. Linden, H. R. Leach, Ludwig A. Ott, B. F. Groat, Forrest Nagler, Robert E. Horton, and Richard Van Vliet. 796.

"Spillway Discharge Capacity of Wilson Dam." Louis G. Puls. (With Discussion.) 316.

PILE DRIVING.

"Construction of the James River Bridge Project." R. C. Wilson and Herbert B. Pope. (With Discussion.) 244.

PILES.

"Construction of the James River Bridge Project." R. C. Wilson and Herbert B. Pope. (With Discussion.) 244.

POWER.

See **ELECTRIC POWER.**

POWER HOUSES.

See **POWER PLANTS.**

POWER PLANTS.

Bibliography of ice in hydro-electric plants. 1145.

"Ice as Affecting —. Final Report of Committee of Power Division." (With Discussion.) 1134.

"Niagara Power." Norman R. Gibson. 1. Discussion: Thomas H. Hogg, Frank M. Williams, Charles Keller, William Kelly, R. D. Johnson, Glen E. Edgerton, F. P. Williams, and Lewis B. Stillwell. 29.

"The Shannon Power Development in the Irish Free State." A. R. C. Markl. (With Discussion.) 597.

POWER STATIONS.

See **POWER PLANTS.**

POWER TRANSMISSION.

"Niagara Power." Norman R. Gibson. (With Discussion.) 1.

RAILROADS.

Railways *versus* waterways: An economic comparison. 877.

"Relation Between Rail and Waterway Transportation": A Symposium. T. Q. Ashburn, L. D. Cornish, and E. A. Hadley. 861. Discussion: Lewis C. Sorrell, R. B. Kittredge, Henry A. Palmer, J. Parker Snow, Baxter L. Brown, Theodore Brent, Samuel O. Dunn, Henry W. Hobbs, and C. W. Kutz. 905.

See also **TUNNELS.**

RAINFALL.

"Frequency and Intensity of Excessive Rainfalls at Boston, Massachusetts." Charles W. Sherman. 951. Discussion: W. W. Horner, C. S. Jarvis, and Milton F. Wagnitz, and Lewis C. Wilcoxen. 961.

"— Characteristics and Their Relation to Soils and Run-Off." C. S. Jarvis. 379. Discussion: George S. Bliss, Brehon B. Somervell, Jesse W. Shuman, Clinton L. Bogert, R. W. Powell, Glen N. Cox, C. R. Pettis, and George C. Danforth. 424.

RATE-MAKING.

"Relation Between Rail and Waterway Transportation": A Symposium. T. Q. Ashburn, L. D. Cornish, and E. A. Hadley. (With Discussion.) 861.

REPORTS OF COMMITTEES.

"A National Reclamation Policy. Report of the Committee of the Irrigation Division." J. B. Lippincott, G. M. Bacon, W. H. Code, J. A. Elliott, John E. Field, C. E. Grunsky, M. C. Hinderlider, Burton Smith, G. E. P. Smith, J. C. Stevens, and R. K. Tiffany. 1303. With Explanatory Statements: J. B. Lippincott, and Elwood Mead. 1307. Discussion: George W. Fuller, W. F. Russell, Nils A. Olsen, L. C. Gray, Frank Adams, M. J. Dowd, C. E. Grunsky, S. T. Harding, H. H. Dare, Charles R. Hedke, Charles Kirby Fox, Rhodes E. Rule, George M. Bacon, G. E. P. Smith, R. K. Tiffany, David Weeks, John A. Widtsoe, F. H. Newell, A. D. Lewis, Joseph Jacobs, F. N. Cronholm, R. I. Meeker, E. B. Debler, and John E. Field. 1328.

"Florida Hurricane. Final Report of the Committee of the Structural Division." H. G. Balcom, D. C. Coyle, F. R. McMillan, Lee H. Miller, and Clyde T. Morris. 1118. Discussion: F. O. Dufour, T. L. Condron, E. W. Stern, and David A. Molitor. 1127.

"Ice as Affecting Power Plants. Final Report of Committee of Power Division." Francis C. Shenehon, John R. Freeman, Daniel W. Mead, Louis C. Sabin, and William T. Walker. 1134. Discussion: C. M. Daily. 1151.

"Impact in Highway Bridges. Final Report of the Special Committee." A. H. Fuller, Arthur R. Eitzen, E. F. Kelley, and F. E. Turneaure. 1089. Discussion: Searcy B. Slack and Llewellyn N. Edwards. 1112.

"Sewage Works Operation and Control. Final Report of the Committee of the Sanitary Engineering Division." C. E. Keefer, W. R. Copeland, A. L. Fales, J. A. Vogleson, and F. H. Waring. 1273.

"Steel Column Research. Second Progress Report of the Special Committee." F. E. Turneaure, C. G. E. Larsson, B. R. Leffler, G. L. Taylor, and S. H. Widdicombe. 1152. Discussion: Henry S. Prichard, E. H. Salmon, and J. H. Griffith. 1255.

RESERVOIRS.

"Baldwin Filtration Plant, Cleveland, Ohio." J. W. Ellms, G. W. Hamlin, A. G. Levy, and J. E. A. Linders. (With Discussion.) 459.

"Evaporation as a Function of Insolation." Burt Richardson. (With Discussion.) 996.

High dams: The viewpoint of the geologist. 149.
"Silting and Life of Southwestern —." R. G. Hemphill. 1060. Discussion: T. U. Taylor. 1073.

RIVER REGULATION.
See **RIVERS.**

RIVERS.
"A National Reclamation Policy. Report of the Committee of the Irrigation Division." With Explanatory Statements: J. B. Lippincott, and Elwood Mead. (With Discussion.) 1303.
Effect of power development on Lake Erie and the Niagara River. 16.
Limitation of diversion of water of Niagara River at Niagara Falls. 14, 36.
"Regulation of Levels, Flow, and Navigation, Niagara River: Summary and Conclusions of Various Studies." George B. Pillsbury. 51. Discussion: Robert E. Horton, Lynn L. Davis, Theron M. Ripley, E. W. Lane, and F. G. Way. 63.
"Silting and Life of Southwestern Reservoirs." R. G. Hemphill. (With Discussion.) 1060.
"The Training Wall Across the Liao Bar in Manchuria." P. N. Fawcett. (With Discussion.) 969.
Ultimate development of Niagara Falls. 22.

ROADS.
Comparative costs of different types of highways in and adjacent to New York City. 1050.
"Highways as Elements of Transportation." Fred Lavis. (With Discussion.) 1020.

RUN-OFF.
"Rainfall Characteristics and Their Relation to Soils and —." C. S. Jarvis. (With Discussion.) 379.

SEWAGE DISPOSAL.
"Sewage Works Operation and Control. Final Report of the Committee of the Sanitary Engineering Division." 1273.

SEWAGE TREATMENT.
See **SEWAGE DISPOSAL.**

SEWAGE WORKS.
See **SEWAGE DISPOSAL.**

SHIP CANALS.
See **CANALS.**

SILT.
"Silting and Life of Southwestern Reservoirs." R. G. Hemphill. (With Discussion.) 1060.

SOILS.
"Rainfall Characteristics and Their Relation to — and Run-Off." C. S. Jarvis. (With Discussion.) 379.

SPILLWAYS.
"Spillway Discharge Capacity of Wilson Dam." Louis G. Puls. (With Discussion.) 316.

STEEL.
"— Column Research. Second Progress Report of the Special Committee." (With Discussion.) 1152.

STRAIN.
"Steel Column Research. Second Progress Report of the Special Committee." (With Discussion.) 1152.

STREAMS.
See **RIVERS.**

STRESS.
"Impact in Highway Bridges. Final Report of the Special Committee." (With Discussion.) 1089.
"Laminated Arch Dams with Forked Abutments." Fred A. Noetzli. (With Discussion.) 533.
"Plastic Flow in Concrete Arches." Lorenz G. Straub. (With Discussion.) 613.
"Steel Column Research. Second Progress Report of the Special Committee." (With Discussion.) 1152.

STRUCTURES, THEORY OF.
"Laminated Arch Dams with Forked Abutments." Fred A. Noetzli. (With Discussion.) 533.
"Plastic Flow in Concrete Arches." Lorenz G. Straub. (With Discussion.) 613.

SURVEYING.
"Adjustment of Transit and Stadia Traverses." Howard S. Rappleye. 232. Discussion: Lowell O. Stewart, Paul H. Underwood, F. H. Peters, and H. Parry. 237.

TEMPERATURE.
"Evaporation as a Function of Insolation." Burt Richardson. (With Discussion.) 996.

TESTING MACHINES.
"Experimental Research on Vibration Dampers and Insulators." Stephen E. Slocum. (With Discussion.) 85.
"Steel Column Research. Second Progress Report of the Special Committee." (With Discussion.) 1152.

TOPOGRAPHY.
Significance of — at a dam site. 151.

TRAFFIC.
— surveys. 1025, *et seq.*
See also **FREIGHT.**

TRAIN RESISTANCE.
Air resistance in Moffat Tunnel. 359.

TRAINING WALLS.
See **JETTIES.**

TRANSPORTATION.
"Factors Governing the Location of Airports." Donald M. Baker. (With Discussion.) 269.
"Highways as Elements of —." Fred Lavis. (With Discussion.) 1020.
Railways *versus* waterways: An economic comparison. 877.
"Relation Between Rail and Waterway —": A Symposium. T. Q. Ashburn, L. D. Cornish, and E. A. Hadley. (With Discussion.) 861.
Waterway — from the viewpoint of operation. 862.

TUNNELS.
"Completion of Moffat Tunnel of Colorado." Clifford Allen Betts. 334. Discussion: J. Vipond Davies, R. H. Keays, and A. A. Eremin. 372.
Cost data, Moffat Tunnel. 369.

UPLIFT (HYDROSTATIC).
"High Dams": A Symposium. A. J. Wiley, D. C. Henny, F. L. Ransome, and J. G. Tripp. (With Discussion.) 129.

VENTILATION.
"Completion of Moffat Tunnel of Colorado." Clifford Allen Betts. (With Discussion.) 334.
— of the Moffat Tunnel. 356.

VIBRATION.
"Experimental Research on — Dampers and Insulators." Stephen E. Slocum. 85. Discussion: J. P. Den Hartog, Francis G. Tatnall, and W. M. Dawley. 106.

WATER.
"Baldwin Filtration Plant, Cleveland, Ohio." J. W. Ellms, G. W. Hamlin, A. G. Levy, and J. E. A. Linders. (With Discussion.) 459.

WATER, FLOW OF, IN OPEN CHANNELS.
Effect of tides and currents on Chesapeake and Delaware Canal. 758.
"Effect of Turbulence on the Registration of Current Meters." David L. Yarnell and Floyd A. Nagler. (With Discussion.) 766.

"Regulation of Levels, Flow, and Navigation, Niagara River: Summary and Conclusions of Various Studies." George B. Pillsbury. (With Discussion.) 51.

WATER, FLOW OF, OVER DAMS AND WEIRS.

"Spillway Discharge Capacity of Wilson Dam." Louis G. Puls. 316. Discussion: Cecil E. Pearce, and J. H. Jones. 330.

WATER LAWS.

"A National Reclamation Policy. Report of the Committee of the Irrigation Division." With Explanatory Statements: J. B. Lippincott, and Elwood Mead. (With Discussion.) 1303.

WATER POWER.

"Niagara Power." Norman R. Gibson. (With Discussion.) 1.

"The Shannon Power Development in the Irish Free State." A. R. C. Markl. 597. Discussion: L. J. Bevan. 612.

WATER STORAGE.

"Silting and Life of Southwestern Reservoirs." R. G. Hemphill. (With Discussion.) 1060.

WATERWAYS.

Railways *versus* —: An economic comparison. 877.

"Relation Between Rail and Waterway Transportation": A Symposium. T. Q. Ashburn, L. D. Cornish, and E. A. Hadley. (With Discussion.) 861.

Waterway transportation from the viewpoint of operation. 862.

WATER-WORKS.

See **FILTRATION.**

WEIRS.

See **WATER, FLOW OF, OVER DAMS AND WEIRS.**

WIND.

"Florida Hurricane. Final Report of the Committee of the Structural Division." (With Discussion.) 1118.

WIND BRACING.

"Florida Hurricane. Final Report of the Committee of the Structural Division." (With Discussion.) 1118.

WIND PRESSURE.

"Florida Hurricane. Final Report of the Committee of the Structural Division." (With Discussion.) 1118.

ZONING OF CITIES.

"City Planning as Related to the Smaller Cities." E. A. Wood. (With Discussion.) 1075.

AUTHOR INDEX

ABBOT, C. G.
Insolation and evaporation. 1013.

ADAMS, FRANK.
A National reclamation policy. 1331.

ALLEN, KENNETH.
Memoir of. 1437.

ALLISON, WILLIAM FRANKLIN.
Memoir of. 1440.

ANDERSON, JUSTIN KENDERDINE.
Memoir of. 1442.

ANGELL, N. H.
Location of airports. 285.

ASH, HENRY CLARKE.
Memoir of. 1443.

ASHBURN, T. Q.
"Waterway Transportation from the Viewpoint of Operation." 862.

ASHMEAD, FRANK MILLIGAN.
Memoir of. 1445.

AXTELL, FRANK FOY.
Memoir of. 1446.

BACON, GEORGE M.
"'A National Reclamation Policy'. Report of the Committee of the Irrigation Division." 1303, 1362.

BAKER, DONALD M.
"Factors Governing the Location of Airports." 269.
Highways as elements of transportation. 1053.

BALCOM, H. G.
"Florida Hurricane. Final Report of the Committee of the Structural Division." 1118.

BALLARD, WILSON T.
James River Bridge project. 262.

BARNES, MORTIMER GRANT.
Memoir of. 1449.

BAUMAN, PAUL.
Laminated arch dams. 583.
Plastic flow in concrete arches. 704.

BENNETT, CHARLES S.
Effect of turbulence on current meters. 796.

BENT, STEDMAN.
Memoir of. 1663.

BERQUIST, AXEL SAMUEL FREDERICK.
Memoir of. 1451.

BETTS, CLIFFORD ALLEN.
"Completion of Moffat Tunnel of Colorado." 334.

BEVAN, L. J.
The Shannon Power Development. 612.

BIGWOOD, BURKE L.
Effect of turbulence on current meters. 811.

BLACK, ALEXANDER LESLIE.
Memoir of. 1452.

BLAND, JOHN CARLISLE.
Memoir of. 1453.

BLISS, GEORGE S.
Rainfall characteristics. 424.

BOGERT, CLINTON L.
Rainfall characteristics. 432.

BOWERS, GEORGE.
Memoir of. 1456.

BRATAGER, STOERK JOHAN.
Memoir of. 1457.

BRENDLINGER, PETER FRANKLIN.
Memoir of. 1459.

BRENT, THEODORE.
Rail and waterway transportation. 919.

BROWN, BAXTER L.
Rail and waterway transportation. 917.

BROWN, EARL I.
"The Chesapeake and Delaware Canal." 716.

BROWN, ROBERT.
Memoir of. 1462.

BURR, GEORGE D.
Location of airports. 283.

BURR, HENRY AMSDEN.
Memoir of. 1463.

BYERS, MAXWELL CUNNINGHAM.
Memoir of. 1465.

CAIN, WILLIAM.
Laminated arch dams. 566.
Memoir of. 1467.

CAMPEN, GEORGE LINDEN.
Memoir of. 1473.

CARSON, WILLIAM WALLER.
Memoir of. 1475.

CHAMBERS, F. T.
The Chesapeake and Delaware Canal. 764.

CHASE, E. SHERMAN.
Baldwin Filtration Plant, Cleveland. 529.

CHATLEY, HERBERT.
Training wall across Liao Bar, Manchuria. 985.

CHILD, STEPHEN.
City planning for smaller cities. 1084.
Location of airports. 279.

CODE, W. H.
"'A National Reclamation Policy'. Report of the Committee of the Irrigation Division." 1303.

COMSTOCK, CHARLES W.
Plastic flow in concrete arches. 694.

CONDRON, T. L.
Effects of Florida hurricane. 1128.

COOLEY, MAURICE WURTZ.
Memoir of. 1477.

COPELAND, W. R.
"Sewage Works Operation and Control. Final Report of the Committee of the Sanitary Engineering Division." 1273.

CORNISH, L. D.
"Railways Versus Waterways: An Economic Comparison." 877.

COX, GLEN N.
Rainfall characteristics. 439.

COYLE, D. C.
"Florida Hurricane. Final Report of the Committee of the Structural Division." 1118.

CREAGER, WILLIAM P.
High dams. 197.

CRONHOLM, F. N.
A National reclamation policy. 1395.

CROSBY, W. W.
Highways as elements of transportation. 1041.
Location of airports. 308.

CUMMINGS, N. W.
Insolation and evaporation. 1017.

DAILY, C. M.
Ice as affecting power plants. 1151.

DANFORTH, GEORGE C.
Rainfall characteristics. 444.

DARE, H. H.
A National reclamation policy. 1343.

DAVIDSON, GEORGE.
Memoir of. 1428.

DAVIES, J. VIPOND.
Completion of Moffat Tunnel. 372.

DAVIS, LYNN L.
Regulation of Niagara River. 75.

DAWLEY, W. M.
Vibration dampers and insulators. 110.

DEBLER, E. B.
A National reclamation policy. 1403.

DEN HARTOG, J. P.
Vibration dampers and insulators. 106.

DODGE, GRENVILLE MELLEN.
Memoir of. 1432.

DOWD, M. J.
A National reclamation policy. 1335.

DOWNER, JAY.
Highways as elements of transportation. 1044.

DUFOUR, F. O.
Effects of Florida hurricane. 1127.

DUNHAM, H. F.
High dams. 195.

DUNN, SAMUEL O.
Rail and waterway transportation. 921.

DURYEA, EDWIN, Jr.
Memoir of. 1478.

EDGERTON, GLEN E.
Niagara power. 44.

EDWARDS, LLEWELLYN N.
Impact in highway bridges. 1112.

EDWARDS, RAYMOND ARDEN.
Memoir of. 1634.

EITZEN, ARTHUR R.
"Final Report of the Special Committee on Impact in Highway Bridges." 1089.

ELLIOTT, J. A.
"'A National Reclamation Policy'. Report of the Committee of the Irrigation Division." 1303.

ELLMS, J. W.
"Baldwin Filtration Plant, Cleveland, Ohio." 459.

EREMIN, A. A.
Completion of Moffat Tunnel. 375.
Laminated arch dams. 589.
Plastic flow in concrete arches. 696.

FALES, A. L.
"Sewage Works Operation and Control. Final Report of the Committee of the Sanitary Engineering Division." 1273.

FAWCETT, P. N.
"The Training Wall Across the Liao Bar in Manchuria." 969.

FEHR, HARRISON ROBERTS, Sr.
Memoir of. 1480.

FELLOWS, PERRY A.
Location of airports. 308.

FELTON, SAMUEL MORSE.
Memoir of. 1481.

FIELD, JOHN E.
"'A National Reclamation Policy'. Report of the Committee of the Irrigation Division." 1303, 1408.

FLORIS, A.
High dams. 200.
Plastic flow in concrete arches. 710.

FOSTER, CLARENCE MARVIN.
Memoir of. 1664.

FOX, CHARLES KIRBY.
A National reclamation policy. 1357.

FOX, WILLIAM J.
Location of airports. 296.

FREEMAN, JOHN R.
"Ice as Affecting Power Plants. Final Report of Committee of Power Division." 1134.

FREEMAN, ROGER MORSE.
Memoir of. 1485.

FULLER, A. H.
"Final Report of the Special Committee on Impact in Highway Bridges." 1089.

FULLER, GEORGE W.
A National reclamation policy. 1328.

FULTON, WILLIAM JAMES.
Memoir of. 1636.

GATES, HORACE DELPHOS.
Memoir of. 1488.

GERDINE, THOMAS GOLDING.
Memoir of. 1490.

GIBSON, NORMAN R.
"Niagara Power." 1.

GILKEY, HERBERT J.
Plastic flow in concrete arches. 699.

GILMORE, THOMAS NIXON.
Memoir of. 1492.

GODFREY, EDWARD.
High dams. 174.
Plastic flow in concrete arches. 685.

GOODRICH, R. D.
Training wall across Liao Bar, Manchuria. 984.

GOODWIN, ARTHUR BATES.
Memoir of. 1637.

GRAY, L. C.
A National reclamation policy. 1331.

GRIFFITH, J. H.
Plastic flow in concrete arches. 688.
Steel column research. 1267.

GRIMM, S. N.
Location of airports. 290.

GROAT, B. F.
Effect of turbulence on current meters. 830.

GROVER, N. C.
Effect of turbulence on current meters. 806.

GRUNSKY, C. E.
"'A National Reclamation Policy'. Report of the Committee of the Irrigation Division." 1303, 1338.

HADLEY, E. A.
"Relation Between Rail and Waterway Transportation." 898.

HALL, BENJAMIN MORTIMER.
Memoir of. 1493.

HAMLIN, G. W.
"Baldwin Filtration Plant, Cleveland, Ohio." 459.

HANNA, F. W.
High dams. 192.
Laminated arch dams. 586.

HARDING, S. T.
A National reclamation policy. 1341.

HART, CHARLES ELLETT.
Memoir of. 1639.

HARVEY, ALFRED ERNEST.
Memoir of. 1496.

HARZA, L. F.
High dams. 198.

HAWGOOD, HARRY.
Memoir of. 1497.

HAZEN, ALLEN.
High dams. 183.
Memoir of. 1500.

HEDKE, CHARLES R.
A National reclamation policy. 1346.

HEMPHILL, ROBERT GRIER.
Memoir of. 1639.
"Silting and Life of Southwestern Reservoirs." 1060.

HENNY, D. C.
"Classification, Selection, and Adaptation of High Dams." 139.

HERRON, JAMES H.
Baldwin Filtration Plant, Cleveland. 528.

HERSCHEL, CLEMENS.
Memoir of. 1419.

HILL, ALBERT BANKS.
Memoir of. 1503.

HILL, EDWARD BAXTER.
Memoir of. 1641.

HILL, EDWIN ALLSTON.
Memoir of. 1642.

HILL, JOHN WILMUTH.
Memoir of. 1506.

HINDERLIDER, M. C.
"'A National Reclamation Policy'. Report of the Committee of the Irrigation Division." 1303.

HOBBS, HENRY W.
Rail and waterway transportation. 932.

HOGELAND, ALBERT HARRISON.
Memoir of. 1509.

HOGG, THOMAS H.
Niagara power. 29.

HOLGATE, HENRY.
Memoir of. 1511.

HORNER, W. W.
Excessive rainfall at Boston, Mass. 961.

HORTON, ROBERT E.
Effect of turbulence on current meters. 840.
Regulation of Niagara River. 63.

HOXIE, RICHARD LEVERIDGE.
Memoir of. 1513.

HOYT, WILLIAM HAUSMER.
Memoir of. 1515.

HUCKABY, MARION COLUMBUS.
Memoir of. 1517.

HUDSON, H. W.
Highways as elements of transportation. 1054.

HUGHES, HECTOR JAMES.
Memoir of. 1518.

JACOBS, JOSEPH.
A National reclamation policy. 1392.

JAKOBSEN, B. F.
High dams. 179.
Laminated arch dams. 568.
Plastic flow in concrete arches. 679.

JARVIS, C. S.
Excessive rainfall at Boston, Mass. 962.
"Rainfall Characteristics and Their Relation to Soils and Run-Off." 379.

JENKS, HARRY N.
Baldwin Filtration Plant, Cleveland. 519.

JENNINGS, JOHN EDWARD.
Memoir of. 1520.

JOHANNESSON, S.
Highways as elements of transportation. 1045.

JOHNSON, R. D.
Niagara power. 43.

JONES, J. H.
Spillway discharge capacity. 332.

JORGENSEN, LARS R.
High dams. 196.
Laminated arch dams. 565.

JUNKERSFELD, PETER.
Memoir of. 1522.

KEAYS, R. H.
Completion of Moffat Tunnel. 374.

KEEFER, C. E.
"Sewage Works Operation and Control. Final Report of the Committee of the Sanitary Engineering Division." 1273.

KELLER, CHARLES.
Niagara power. 36.

KELLEY, E. F.
"Final Report of the Special Committee on Impact in Highway Bridges." 1089.

KELLY, WILLIAM.
Niagara power. 41.

KIMBALL, H. H.
Insolation and evaporation. 1012.

KINSLEY, THOMAS PEARSON.
Memoir of. 1524.

KITTREDGE, R. B.
Rail and waterway transportation. 911.

KNOWLES, MORRIS.
Location of airports. 281.

KOCH, O. H.
City planning for smaller cities. 1081.

KREUTZER, GEORGE CHARLES.
Memoir of. 1526.

KRYNINE, D. P.
Highways as elements of transportation. 1046.
Insolation and evaporation. 1013.

KUTZ, C. W.
Rail and waterway transportation. 934.

KWONG, KING YANG.
Memoir of. 1528.

LANE, E. W.
Regulation of Niagara River. 80.

LARSSON, C. G. E.
"Second Progress Report of the Special Committee on Steel Column Research." 1152.

LAVIS, FRED.
"Highways as Elements of Transportation." 1020.

LEACH, H. R.
Effect of turbulence on current meters. 816.

LEFFLER, B. R.
"Second Progress Report of the Special Committee on Steel Column Research." 1152.

LEVY, A. G.
"Baldwin Filtration Plant, Cleveland, Ohio." 459.

LEWIS, A. D.
A National reclamation policy. 1388.

LEWIS, HAROLD M.
Highways as elements of transportation. 1047.

LINDEN, C.
Effect of turbulence on current meters. 814.

LINDERS, J. E. A.
"Baldwin Filtration Plant, Cleveland, Ohio." 459.

LIPPINCOTT, J. B.
"A National Reclamation Policy: Explanatory Statement Regarding Report of the Committee of the Irrigation Division." 1307.
"'A National Reclamation Policy'. Report of the Committee of the Irrigation Division." 1303.

LONG, EUGENE McLEAN.
Memoir of. 1531.

LOTTER, HENRY HOWELL.
Memoir of. 1533.

LUDLOW, JACOB LOTT.
Memoir of. 1536.

LUNDIE, JOHN.
Memoir of. 1538.

MacKAY, HENRY MARTYN.
Memoir of. 1541.

MACLAY, WILLIAM WALTER.
Memoir of. 1543.

McCARTHY, GEORGE ARNOLD.
Memoir of. 1545.

McCLINTOCK, WILLIAM EDWARD.
Memoir of. 1546.

McCOY, LAURENCE FRANCIS.
Memoir of. 1549.

McDONALD, JOHN ALEXANDER.
Memoir of. 1550.

McGREW, ANSON BURLINGAME.
Memoir of. 1552.

McMILLAN, F. R.
"Florida Hurricane. Final Report of the Committee of the Structural Division." 1118.

MANDIGO, CLARK ROGERS.
Memoir of. 1553.

MANSON, MARSDEN.
Memoir of. 1554.

MARKL, A. R. C.
"The Shannon Power Development in the Irish Free State." 597.

MARSH, J. OGDEN.
Location of airports. 286.

MARTIN, CHARLES CYRIL.
Memoir of. 1556.

MEAD, DANIEL W.
"Ice as Affecting Power Plants. Final Report of Committee of Power Division." 1134.

MEAD, ELWOOD.
"A National Reclamation Policy: Economic Aspects of Federal Reclamation." 1321.

MEEKER, R. I.
A National reclamation policy. 1401.

MELAN, J.
Plastic flow in concrete arches. 698.

MILLER, LEE H.
"Florida Hurricane. Final Report of the Committee of the Structural Division." 1118.

MINOR, SHELDON ELTON.
Memoir of. 1558.

MITCHELL, HENRY.
Memoir of. 1560.

MOLITOR, DAVID A.
Effects of Florida hurricane. 1131.

MONCRIEFF, JOHN MITCHELL.
Memoir of. 1561.

MORRIS, CLYDE T.
"Florida Hurricane. Final Report of the Committee of the Structural Division." 1118.

MOULTON, GUY.
Memoir of. 1563.

MUCKLESTON, H. B.
Laminated arch dams. 573.

MUNSTER, ANDREAS WENDELBO.
Memoir of. 1565.

NAGLER, FLOYD A.
"Effect of Turbulence on the Registration of Current Meters." 766.

NAGLER, FORREST.
Effect of turbulence on current meters. 834.

NETTLETON, G. H.
Effect of turbulence on current meters. 812.

NEWELL, F. H.
A National reclamation policy. 1383.

NICKERSON, RALPH WALDO.
Memoir of. 1566.

NOETZLI, FRED A.
High dams. 171.
"Laminated Arch Dams with Forked Abutments." 533.

NOLEN, JOHN.
Location of airports. 288.

NORCROSS, ARCHER R.
Location of airports. 287.

NORMAN, GEORGE H.
Memoir of. 1568.

NORTON, GEORGE HARVEY.
Memoir of. 1569.

O'CONNOR, CORNELIUS JOSEPH.
Memoir of. 1644.

OKAZAKI, B.
Training wall across Liao Bar, Manchuria. 987.

OLSEN, NILS A.
A National reclamation policy. 1331.

OTT, LUDWIG A.
Effect of turbulence on current meters. 826.

PALMER, HENRY A.
Rail and waterway transportation. 914.

PARKS, CHARLES WELLMAN.
Memoir of. 1571.

PARRY, H.
Transit and stadia traverses. 241.

PARSONS, H. de B.
High dams. 184.

PARTRIDGE, JOHN F.
Location of airports. 296.

PEARCE, CECIL E.
Spillway discharge capacity. 330.

PETERS, F. H.
Transit and stadia traverses. 240.

PETTIS, C. R.
Rainfall characteristics. 440.

PILLSBURY, GEORGE B.
"Regulation of Levels, Flow, and Navigation, Niagara River: Summary and Conclusions of Various Studies." 51.

POLHEMUS, JAMES SUYDAM.
Memoir of. 1575.

POPE, HERBERT B.
"Construction of the James River Bridge Project." 244.

POPE, JAMES BOND.
Memoir of. 1576.

PORTER, GEORGE FREDERICK.
Memoir of. 1577.

PORTER, WALDO JUSTIN BICKLE.
Memoir of. 1645.

POWELL, R. W.
Rainfall characteristics. 432.

PRATT, WILLIAM ABBOTT.
Memoir of. 1579.

PRICHARD, HENRY S.
Steel column research. 1255.

PROBST, E.
Plastic flow in concrete arches. 703.

PUGH, MARSHALL ROGERS.
Memoir of. 1581.

PULS, LOUIS G.
"Spillway Discharge Capacity of Wilson Dam." 316.

RAITT, JOHN WILLIAM.
Memoir of. 1584.

RANSOME, F. L.
"High Dams: The Viewpoint of the Geologist." 149.

RAPPLEYE, HOWARD S.
"Adjustment of Transit and Stadia Traverses." 232.

RAY, F. G.
Regulation of Niagara River. 81.

REED, WILLIAM BOARDMAN.
Memoir of. 1585.

RHODES, CLAUDE IRVIN.
Memoir of. 1586.

RICHARDSON, BURT.
"Evaporation as a Function of Insolation." 996.

RIFFLE, FRANKLIN.
Memoir of. 1587.

RIPLEY, H. C.
Training wall across Liao Bar, Manchuria. 986.

RIPLEY, THERON M.
Regulation of Niagara River. 79.

RIX, EDWARD AUSTIN.
Memoir of. 1590.

RODD, THOMAS.
Memoir of. 1592.

RODE, HANS HENRIK.
Memoir of. 1594.

ROGERS, JOSEPH WARREN.
Memoir of. 1646.

ROHWER, CARL.
Effect of turbulence on current meters. 797.

ROSTOCK, JOHN HENRY.
Memoir of. 1595.

ROUSSEAU, HARRY HARWOOD.
Memoir of. 1597.

RULE, RHODES E.
A National reclamation policy. 1360.

RUSSELL, RICHARD LORD.
Memoir of. 1601.

RUSSELL, W. F.
A National reclamation policy. 1329.

SABIN, LOUIS C.
"Ice as Affecting Power Plants. Final Report of Committee of Power Division." 1134.

ST. JOHN, R. U.
Location of airports. 292.

SALMON, E. H.
Steel column research. 1258.

SAMUEL, GEORGE FREDERICK.
Memoir of. 1603.

SCRIPKO, NICHOLAS ALEXANDER.
Memoir of. 1649.

SEAMAN, WILLIAM LAURIE.
Memoir of. 1650.

SHAFFER, IVAN OSCAR.
Memoir of. 1604.

SHARTS, STANLEY RUSH.
Memoir of. 1605.

SHENEHON, FRANCIS C.
"Ice as Affecting Power Plants. Final Report of Committee of Power Division." 1134.

SHERMAN, CHARLES W.
"Frequency and Intensity of Excessive Rainfalls at Boston, Massachusetts." 951.

SHUMAN, JESSE W.
Rainfall characteristics. 426.

SIMPSON, ERASTUS ROLAND.
Memoir of. 1652.

SLACK, SEARCY B.
Impact in highway bridges. 1112.

SLOCUM, STEPHEN E.
"Experimental Research on Vibration Dampers and Insulators." 85.

SMITH, BRONSON HASBROUCK.
Memoir of. 1606.

SMITH, BURTON.
"'A National Reclamation Policy'. Report of the Committee of the Irrigation Division." 1303.

SMITH, E. K.
Location of airports. 309.

SMITH, G. E. P.
"'A National Reclamation Policy'. Report of the Committee of the Irrigation Division." 1303, 1364.

SNOW, J. PARKER.
Rail and waterway transportation. 915.

SOMERVELL, BREHON B.
Effect of turbulence on current meters. 800.
Rainfall characteristics. 424.

SORRELL, LEWIS C.
Rail and waterway transportation. 905.

STAEHLE, G. C.
Plastic flow in concrete arches. 709.

STARKIE, JOHN L.
City planning for smaller cities. 1080.

STERN, E. W.
Effects of Florida hurricane. 1128.

STEVENS, J. C.
"'A National Reclamation Policy'. Report of the Committee of the Irrigation Division." 1303.

STEWART, LOWELL O.
Transit and stadia traverses. 237.

STILLWELL, LEWIS B.
Niagara power. 48.

STOLL, HAROLD DAVID.
Memoir of. 1653.

STOREY, FRANKLIN STEVENS.
Memoir of. 1655.

STRAUB, LORENZ G.
"Plastic Flow in Concrete Arches." 613.

STUART, FRANCIS LEE.
"The Engineer's Growing Civic Responsibilities": Address at the Annual Convention, Tacoma, Washington, July 8, 1931. 1294.

STUBBS, LINTON WADDELL.
Memoir of. 1608.

SWITZER, F. G.
High dams. 169.

TATNALL, FRANCIS G.
Vibration dampers and insulators. 108.

TAYLOR, G. L.
"Second Progress Report of the Special Committee on Steel Column Research." 1152.

TAYLOR, HARRY.
Memoir of. 1609.

TAYLOR, T. U.
Silting of Southwestern reservoirs. 1073.

THEBAN, JOHN GERARD.
Memoir of. 1656.

TIFFANY, R. K.
"'A National Reclamation Policy'. Report of the Committee of the Irrigation Division." 1303, 1371.

TIFFANY, WALTER CHECKLEY.
Memoir of. 1658.

TRIBUS, LOUIS LINCOLN.
Memoir of. 1612.

TRIPP, J. G.
"Construction Methods and Plant Layout at Coolidge Dam, in Arizona." 159.

TURNEAURE, F. E.
"Final Report of the Special Committee on Impact in Highway Bridges." 1089.
"Second Progress Report of the Special Committee on Steel Column Research." 1152.

TURNER, C. A. P.
High dams. 188.
Plastic flow in concrete arches. 682.

TURNER, EDMUND KIMBALL.
Memoir of. 1614.

UNDERWOOD, PAUL H.
Transit and stadia traverses. 237.

VAN KEUREN, CHARLES AUGUSTUS.
Memoir of. 1615.

VAN LOAN, S. M.
Baldwin Filtration Plant, Cleveland. 519.

VAN SANT, ROBERT LAWRENCE.
Memoir of. 1616.

VAN VLIET, RICHARD.
Effect of turbulence on current meters. 854.

VAN ZILE, HARRY LEE.
Memoir of. 1617.

VAUGHAN, JAMES RUSSELL.
Memoir of. 1662.

VINCENT, EDWARD FRANKLIN.
Memoir of. 1619.

VOGLESON, J. A.
"Sewage Works Operation and Control. Final Report of the Committee of the Sanitary Engineering Division." 1273.

VOGT, FREDRIK.
Laminated arch dams. 574.
Plastic flow in concrete arches. 699.

WAGNITZ, MILTON F.
Excessive rainfall at Boston, Mass. 963.

WALKER, WILLIAM T.
"Ice as Affecting Power Plants. Final Report of Committee of Power Division." 1134.

WARING, F. H.
"Sewage Works Operation and Control. Final Report of the Committee of the Sanitary Engineering Division." 1273.

WEBB, WALTER LORING.
Highways as elements of transportation. 1042.

WEBSTER, ALBERT LOWRY.
Memoir of. 1621.

WEBSTER, GEORGE SMEDLEY.
Memoir of. 1423.

WEEKS, DAVID.
A National reclamation policy. 1373.

WELLS, R. C.
Insolation and evaporation. 1016.

WENDT, EDWIN F.
Highways as elements of transportation. 1054.

WERNER, P. WILHELM.
High dams. 186.
Laminated arch dams. 572.

WHITAKER, WILLIAM FRANKLIN.
Memoir of. 1660.

WHITFIELD, JAMES EDWARD.
Memoir of. 1622.

WIDDICOMBE, S. H.
"Second Progress Report of the Special Committee on Steel Column Research." 1152.

WIDTSOE, JOHN A.
A National reclamation policy. 1381.

WIGHT, FRANK CLINTON.
Memoir of. 1623.

WILCOXEN, LEWIS C.
Excessive rainfall at Boston, Mass. 963.

WILEY, A. J.
"Past Experience with High Dams and Outlook for the Future." 130.

WILLIAMS, FRANK MARTIN.
Memoir of. 1626.
Niagara power. 34.

WILLIAMS, F. P.
Niagara power. 47.

WILLIAMS, WILLIAM FISH.
Memoir of. 1628.

WILLOUGHBY, WILLIAM.
James River Bridge project. 266.

WILSON, R. C.
"Construction of the James River Bridge Project." 244.

WITT, CARLTON CARPENTER.
Memoir of. 1630.

WOOD, CLYDE M.
Location of airports. 311.

WOOD, E. A.
"City Planning as Related to the Smaller Cities." 1075.

WRIGHT, JOSEPH.
High dams. 205.

YARNELL, DAVID L.
"Effect of Turbulence on the Registration of Current Meters." 766.

YEATMAN, MORGAN EDWARD.
Memoir of. 1632.

YOLTON, ROBERT ELGENE.
Memoir of. 1661.

YOUNG, SAMUEL McCAIN.
Memoir of. 1633.